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STATE OF OHIO

THE MIAMI CONSERVANCY DISTRICT

Rainfall and Runoff in the Miami Valley

BY

IVAN E. HOUK

District Hydrographer; Assoc. M. Am. Soc. C. E.

TECHNICAL REPORTS

Part VIII

DAYTON, OHIO

1921

THE MIAMI CONSERVANCY DISTRICT
DAYTON, OHIO

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PREFATORY NOTE

This volume is the eighth of a series of Technical Reports issued in connection with the planning and execution of the notable system of flood protection works now being built by the Miami Conservancy District.

The Miami Valley, which forms a part of the large interior plain of the central United States and comprises about 4000 square miles of gently rolling topography in southwestern Ohio, is one of the leading industrial centers of the country. Out of the great flood of March, 1913, which destroyed in this valley alone over 360 lives and probably more than $100,000,000 worth of property, there resulted an energetic movement to prevent a recurrence of such a disaster by protecting the entire valley by one comprehensive project. The Miami Conservancy District, established in June, 1915, under the newly enacted Conservancy Act of Ohio, became the agency for securing this protection. On account of the size and character of the undertaking, the plans of the District have been developed with more than usual care.

A report of the Chief Engineer, submitting a plan for the protection of the District from flood damage, was printed in March, 1916, in three volumes of about 200 pages each. After various slight modifications, this report was adopted by the board of directors as the Official Plan of the District, and was republished in May, 1916, under the latter title. This plan for flood protection includes the building of five earth dams across the valleys of the Miami River and its tributaries to form retarding basins, and the improvement of several miles of river channel within the towns and cities of the valley. It is estimated that the dams will contain nearly 8,500,000 cubic yards of earth; that their outlet structures will contain nearly 200,000 cubic yards of concrete; that the river channel improvements will involve the excavation of nearly 5,000,000 cubic yards; and that the whole project will cost about $35,000,000.

At the time of the publication of this volume the flood control works are about three-fourths completed. The Germantown dam and a considerable portion of the channel improvement

work are entirely finished, and the remaining dams and channel work are rapidly approaching completion.

In order to plan the project intelligently many thorough investigations and researches had to be carried out, the results of which have proved of great value to the District and will also, it is believed, be of widespread use to the whole engineering profession. To make the results of these studies available to the residents of the State and to the technical world at large, the District is publishing a series of Technical Reports containing all data of permanent value relating to the history, investigations, design, and construction of the flood prevention works.

The following list shows the titles of the reports published to date and the price at which they may be purchased.

Part I.—The Miami Valley and the 1913 flood, by A. E. Morgan, 1917, 125 pages, 44 illustrations; 50 cents.

Part II.—History of the Miami flood control project, by C. A. Bock, 1918; 196 pages, 41 illustrations; 50 cents.

Part III.—Theory of the hydraulic jump and backwater curves, by S. M. Woodward. Experimental investigation of the hydraulic jump as a means of dissipating energy, by R. M. Riegel and J. C. Beebe, 1917; 111 pages, 88 illustrations; 50 cents.

Part IV.—Calculation of flow in open channels, by I. E. Houk, 1918; 283 pages, 79 illustrations; 75 cents.

Part V.—Storm rainfall of eastern United States, by the engineering staff of the District, 1917; 310 pages, 114 illustrations; 75 cents.

Part VI.—Contract forms and specifications, by the engineering staff of the District, 1918, 192 pages, 3 folding plates, and index; 50 cents.

Atlas of selected contract and information drawings to accompany Part VI; 139 plates, 11 by 15 inches; $1.50.

Part VII.—Hydraulics of the Miami flood control project, by S. M. Woodward, 1920; 344 pages, 126 illustrations; $1.00.

Part VIII.—Rainfall and runoff in the Miami Valley, by I. E. Houk, 1921; 236 pages, 51 illustrations; 75 cents.

Technical reports on the following subjects are contemplated.

Laws relating to flood control.

Structural design, construction plant and methods.

Methods of appraising property benefits and damages.

Orders for Technical Reports should be sent to:

The Miami Conservancy District, Dayton, Ohio.

Arthur E. Morgan,
Chief Engineer.

Dayton, Ohio,
Jan. 1, 1921.

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CHAPTER I.—INTRODUCTION

GENERAL

The purpose of this report is to present to the engineering profession the results of rainfall and runoff investigations carried on in connection with the Miami flood control project.

When an engineering examination of the Miami Valley was begun, immediately after the great flood of March, 1913, in order to determine the best plan for preventing damage by future floods, an investigation of rainfall and runoff conditions was naturally one of the first lines of attack. It was recognized at the start that a knowledge of rainfall and runoff would be essential in determining the size of the flood to be provided for, in the design of the flood protection works, and in the assessment of the benefits and damages which would result from the construction of the works, as well as in the many other problems which probably would be encountered as the development of the plans proceeded. However, as the work progressed and as the magnitude of the problem became apparent, the importance of collecting such data became even more pronounced than had been originally anticipated. Consequently the collection of rainfall and runoff records and the studies of rainfall and runoff relations were more or less gradually expanded during the first few years of the work.

While there were several rainfall stations in the Miami Valley at the time of the 1913 flood, there were but three river gages, one at Piqua, one at Dayton, and one at Hamilton. The work of establishing additional stations was begun, in cooperation with the U. S. Weather Bureau, almost immediately; and within a few months daily records of rainfall and river stages and periodic measurements of discharge were being obtained at several stations on the Mad and Stillwater Rivers, at Germantown on Twin Creek, and at several additional places on the Miami River. Arrangements were also made with the various observers for special readings of river gages during flood periods. The number of stations and the amount of flood data

being secured, was increased from time to time as the work progressed, as will be described in detail later.

Extensive hydrographic surveys of the 1913 flood in the Miami Valley, and investigations of the rainfall over the valley during that storm, were carried on during the summer and fall of 1913. Studies of the relation of the flood runoff to the storm rainfall were made as soon as the data was available. Similar studies for subsequent floods were made from time to time as the floods occurred.

As a practical aid in the study of the relation of runoff to rainfall, a number of small experimental plats were established at Moraine Park, about five miles south of Dayton, where rainfall and surface runoff could be measured on varying slopes and with varying soil conditions, as well as the rapidity and depth of soil saturation caused by different rains. After about four and a half years of records had been secured experiments were undertaken, using garden sprinkling cans to reproduce rainfall effects, in an effort to develop a method by which rainfall and runoff relations could be determined for a given watershed without waiting the comparatively long time required for the collection of sufficient data from natural rainfall. The results obtained were so suggestive that similar plats were established at the Taylorsville Dam where data could be obtained on different types of soil.

SCOPE OF THIS REPORT

Chapter II describes the rainfall and runoff records obtained in the Miami Valley. The records available at the time of the 1913 flood, the stations established since that time, the records secured at the various stations, the gages in use, and the methods of measurement are all discussed in detail. The actual records are not reproduced since the more valuable data is being published elsewhere. However, the places of publication, the particular records being published, and the manner in which the unpublished data may be secured are fully described.

Chapter III takes up the rainfall, runoff, and soil moisture data secured on the small experimental plats at Moraine Park. The records are given in full, in tables and diagrams, and are discussed in detail. The effects of variations in rainfall intensity and in soil moisture content on the surface runoff are taken up, as are also the total rainfall, runoff, and retention during storm

periods. A summary of the principal conditions shown by the data is given at the end of the chapter.

Chapter IV is devoted to the sprinkling experiments at Moraine Park and Taylorsville. The results are shown graphically, by means of mass curves. Summaries of the more important data are given in tabular form. An interesting relation was found to exist between rates of rainfall, runoff, and retention when the surface soil is saturated. The total rainfall, runoff, and retention during the various experiments, as well as the rates, are discussed in detail; and some data is given regarding the intensity and duration of precipitation before surface runoff begins.

In chapter V the monthly, seasonal, and annual rainfall, runoff, and retention throughout the Miami Valley are taken up. Annual conditions in the different drainage areas are shown by means of tables and diagrams. Monthly and seasonal conditions are discussed only for the drainage area above Dayton since the records available for the other stations are of comparatively short duration. A method of studying the hydrology of a valley by means of mass curves is shown, using the data for the drainage area of Mad River above Wright as an example. Discussions of the proportions of ground water runoff and flood runoff are included for the Stillwater, Mad, and Miami Rivers, and Buck Creek.

Chapter VI discusses the rainfall and runoff during the great flood of March, 1913. The data is shown by means of maps and diagrams, but the complete station records are not included. The distribution of the rainfall as regards time as well as drainage area, the characteristics of the flood hydrographs, and the relation of the flood runoff to the storm rainfall are discussed.

Chapter VII takes up the studies of rainfall and runoff which have been made for floods occurring since March, 1913. The total rainfall, runoff, and retention during flood periods; the maximum rates of rainfall and runoff; and the maximum values of retention are given in tabular form and are described in the text. Data is also included relating to storage in stream channels and on the ground, and to precipitation intercepted by trees. Brief descriptions of the various floods are given but the detailed rainfall and runoff records are not included.

Chapter VIII contains a brief description of the flood forecasting work of the District and of the methods used in making the forecasts.

ACKNOWLEDGMENTS

Acknowledgments are due the officials of the United States Weather Bureau, both in Washington and in Dayton, for their cooperation in establishing river and rainfall gaging stations and in maintaining records. All of the rainfall data used in this report except that obtained in the experimental work at Moraine Park, was secured from the Weather Bureau records. Acknowledgments are also due the U. S. Geological Survey, the U. S. Bureau of Soils, and other governmental bureaus.

The engineering organization engaged on the Miami flood control project up to the time construction began was described in detail in an earlier report.* The investigations described in this volume were conducted by the writer assisted at different times by G. N. Burrell, H. W. Wesle, B. H. Petty, H. R. Daubenspeck, F. E. Davis, and others. Professor S. M. Woodward in his capacity as consulting engineer for the District has been frequently consulted. The work has been outlined and supervised at all times by Arthur E. Morgan, chief engineer.

THE MIAMI VALLEY

Rainfall and runoff conditions vary so widely with variations in geology, topography, and climate that it seems pertinent to give a brief description of the Miami Valley.

As may be seen by referring to figure 1 the Miami River flows in a southwesterly direction through southwestern Ohio, entering the Ohio River at the Indiana and Ohio state line. It drains a rather fan shaped area of about 4000 square miles lying almost wholly in Ohio. The Whitewater River which joins the Miami near its mouth and which drains an area of about 1400 square miles lying almost entirely in Indiana, has not been shown since it is not affected by the works of the Miami Conservancy District.

The Miami River is about 163 miles in length. Its drainage basin, which includes parts of 15 counties, measures about 120 miles on the longer axis and about 70 on the shorter. The more important tributaries below Dayton, following northward up the west side of the Miami, are: Indian Creek, emptying just above Venice; Four Mile Creek, a flashy stream entering just above

*History of the Miami Flood Control Project, by C. A. Bock, Technical Reports, Part II, The Miami Conservancy District, Dayton, Ohio, 1918, page 115.

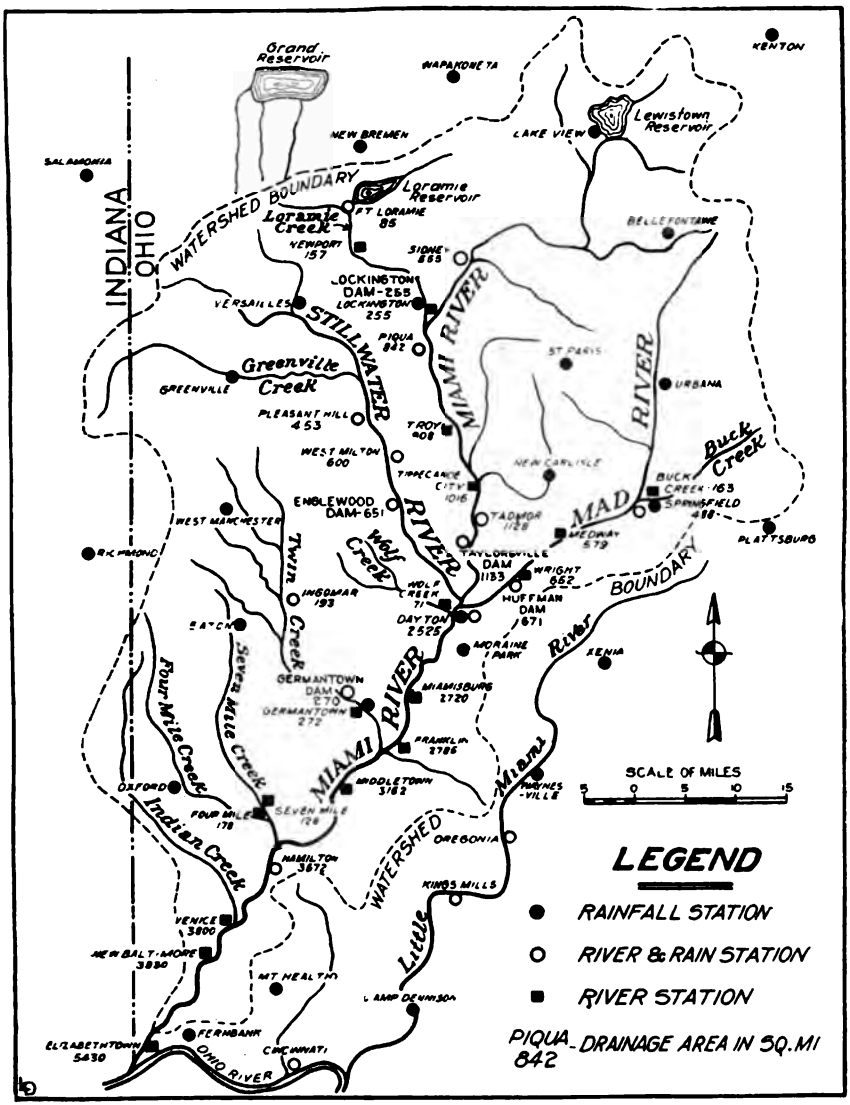


FIG. 1.—MAP OF MIAMI RIVER DRAINAGE AREA SHOWING GAGING STATIONS.

The drainage areas above the river stations, in square miles, are shown by the numbers placed under the names of the stations.

Hamilton; and Twin Creek, with its outlet just below Franklin. Seven Mile Creek flows into Four Mile Creek just above its junction with the Miami. Four streams, the Miami, Mad, and Stillwater Rivers, and Wolf Creek unite within the city limits

of Dayton. Just above Piqua, about 27 miles north of Dayton, the Miami is joined by Loramie Creek. Greenville Creek enters the Stillwater River from the west a few miles above Pleasant Hill. Buck Creek joins the Mad River from the east at Springfield.

The topography of the Miami Valley may be described as gently rolling with the general elevations of the uplands varying from about 800 feet above sea level near the mouth of the river to about 1100 feet near the headwaters. The slopes are comparatively flat near the headwaters but increase more or less gradually toward the southwest, being comparatively abrupt near the Ohio River. Except for its southernmost portion the entire basin bears evidence of having been covered by ice during the glacial period. The preglacial valleys carved in the limestone formations and the crests of the preglacial hills have been almost entirely obliterated by the ice. The Miami River and its principal tributaries flow in the partially filled valleys in comparatively insignificant channels.

The Miami River has a drop of about 2 feet per mile in the first 30 miles, and of about 3.3 feet throughout the major portion of its course. The Stillwater and Mad Rivers are somewhat steeper, the former sloping at a rate of about 4 feet per mile and the latter at a rate of about 6 feet. The smaller tributaries are still steeper, the slopes increasing as we proceed downstream. Twin, Four Mile, and Seven Mile Creeks are noted for the suddenness with which they rise and the short duration of their flood stages. The rolling topography, together with the fan shaped arrangement of the larger tributaries, present singularly favorable conditions for quick collection of storm runoff and the formation of high flood crests.

RAINFALL AND RUNOFF RELATIONS

Rainfall and runoff relations have been studied extensively during recent years. While more or less progress has been made, in methods of investigation as well as in final results, the problem is still far from a complete solution. Early investigators frequently tried to express runoff as a percentage of rainfall; more recently runoff has been considered as a residue remaining after the various losses are supplied. This was undoubtedly a step in advance, since runoff is, essentially, a residue remaining after the demands of evaporation, transpiration, and deep seepage are filled. However, these quanti-

ties, themselves, are so variable and are affected by so many factors that no simple accurate formulas for their calculation can be developed. It must be recognized that there is no simple relation between rainfall and runoff and that runoff can be determined from rainfall within narrow limits, only after very careful study of all existing data by experienced hydraulic engineers. Even then the results may be considerably in error.

This volume does not take up in a general way the factors affecting either rainfall or runoff. Neither does it give any general review of the work of other investigators. For such matters the reader is referred to the original publications and the more recent works on hydrology, listed in the bibliography at the end of the book. The purpose of this report, as before mentioned, is to present the results of the investigations which have been carried on in connection with this project. These were necessarily confined more to studies of actual rainfall and runoff conditions in this valley than to the general laws affecting such phenomena. It will be valuable, however, to point out briefly the more general conclusions regarding rainfall and runoff relations in the Miami Valley, which seem to be justified by the data contained herein.

The Moraine Park records taken up in chapter III show:

1. That variations in surface slope are of much less importance as affecting runoff than are variations in vegetable cover.

2. That intensity of precipitation has an important effect on the occurrence and amount of surface runoff.

3. That during the summer months rainfall seldom percolates to such depths that it is not raised again by capillarity or by root action and evaporated or transpired back into the atmosphere.

4. That storage in the surface soil, filled during winter rains, furnishes about 5 inches of water to the summer evaporation and transpiration requirements.

The sprinkling experiments described in chapter IV show:

5. That the rate of surface runoff increases as the rate of precipitation increases, the former being directly proportional to the latter when the surface soil is saturated.

6. That the rate of percolation, when the surface soil is saturated, increases as the rate of rainfall increases, the variation being according to a straight line equation and the rate of

increase being proportionally greater for loose loamy soils than for heavy clay soils.

7. That cultivation has a relatively important effect in reducing the amount of surface runoff.

The studies of annual runoff given in chapter V show:

8. That annual evaporation, including transpiration, is not constant, but varies with the seasonal distribution and amount of rainfall, as well as with other meteorological factors, the variation for the drainage area above Dayton being slightly greater than the variation in annual rainfall.

9. That annual runoff is much more variable than either annual rainfall or annual evaporation.

10. That annual surface, or flood, runoff is much more variable than the annual low water, or ground water, runoff, the variations in total annual runoff being caused primarily by the variations in surface runoff.

11. That the character and condition of the soil influences runoff to a greater extent than has been generally recognized, and next to rainfall is the most important factor affecting runoff.

The studies of storm rainfall and flood runoff taken up in chapter VII show:

12. That the total retention during similar storms is greater in the summer than in the winter.

Some of the above conclusions are simply confirmations of recognized laws. Others, however, such as number 6, relating to the rate of percolation, are new.

It is believed that the sprinkling method of investigating rainfall and runoff relations, described in chapter IV, offers distinct possibilities; and it is hoped that other investigators will try this out in different parts of the country and on different soils. Of course whenever sufficient funds are available it would be desirable to install lysimeters so that the percolation can be directly measured. Such experiments might be taken up by senior or graduate students at the various technical colleges. Persons desiring to pursue the matter might secure rainfall and runoff data, similar to that contained in chapter IV, for different soils, at different seasons of the year, with different surface coverings, and in different stages of cultivation. Data might also be secured on the intensity and duration of precipitation which will cause surface runoff to begin on different soils, covered with different growing crops

in different stages of growth. Engineers engaged on projects requiring a knowledge of rainfall and runoff relations can probably secure valuable information in a few months by such methods.

Better information regarding variations in soil absorption, percolation, evaporation, and transpiration caused by variations in soil texture, soil moisture, temperature, rainfall, surface conditions, and so forth, will be of material assistance in studies of flood runoff and storm rainfall. Probably future studies of the relation between rainfall and runoff will give better results if the flood runoff and low water runoff are considered separately, even though the division between the two may have to be more or less arbitrarily made. This should be especially true where the flood runoff constitutes a large proportion of the total and is as variable as it is in the Miami Valley.

Records of total rainfall and runoff should be published whenever possible, since such information always is valuable to hydraulic engineers. From a large accumulation of such records certain generalizations sometimes can be made. Where the records are of long duration they furnish valuable data for studies of abnormally dry and wet years as well as of the general variations in total amounts. However, such data is of limited application in that it includes the cumulative effects of the different fundamental laws affecting these phenomena, and it is doubtful whether a detailed study of such records will indicate very definite relations between the two which can be blindly applied to other drainage areas.

CHAPTER II.—RAINFALL AND RUNOFF RECORDS

RECORDS PRIOR TO 1913

In May, 1913, when the Miami Valley flood prevention surveys were begun, the United States Weather Bureau maintained the only gaging stations within the Miami River drainage area above the mouth of the Whitewater. There were eight of these, located at Springfield, Urbana, Bellefontaine, Sidney, Piqua, Greenville, Dayton, and Hamilton. A few additional stations, also maintained by the Weather Bureau, were located just outside the valley at Kings Mills, Waynesville, Plattsburg, Kenton, Wapakoneta, and New Bremen, Ohio, and at Richmond and Salamonia, Indiana.

Daily rainfall records were being secured at all places. At some stations the records had been taken for comparatively long periods of time. At Dayton, for instance, continuous records had been taken since November, 1882; and at Urbana, during the period from January, 1852, to April, 1878, and after January, 1896. At Greenville, continuous records had been taken since February, 1886.

Rainfall records were also available at a few stations which had been established, and later discontinued, by the Weather Bureau. Among these might be mentioned the 10-year records at Bethany and Oxford, Butler County, and at New Paris, Preble County, and the 40-year record at Jacksonburg, Butler County.

However, the river gage records were much fewer in number. Of the above mentioned eight stations within the Miami Valley, river records were being secured at only three, Piqua, Dayton, and Hamilton. At Piqua, flood stages had been observed from January 1, 1907, to December 31, 1910, and daily stages after January 1, 1911. At Dayton, continuous daily stages had been observed beginning December 22, 1892. At Hamilton, flood stages had been observed from November 15, 1904, to February 28, 1910, and daily stages beginning with March 1, 1910.

The Water Resources Branch of the U. S. Geological Survey had maintained a river station on Mad River, about four miles west of Springfield, from December 31, 1903, to March 31, 1906; during which time they secured daily gage readings, except for

certain short periods during the winter months, and made several measurements of discharge. They had also developed a satisfactory rating curve for the channel at Hamilton, for stages up to about twenty feet, and had made a number of low water discharge measurements at Dayton.

In addition to the above, a few records of maximum flood heights at Dayton were available in the early histories and newspapers of the Miami Valley.

Dayton was made a regular Weather Bureau station in August, 1911. From that time the usual meteorological records taken at such stations were available for Dayton, including the automatic records of sunshine, wind velocity, wind direction, and precipitation, made by the triple register, as well as the records of barometric pressure, relative humidity, temperature, and the like.

STATIONS ESTABLISHED SINCE 1913

The river records being secured in 1913 were, of course, inadequate for the flood prevention studies. While the rainfall records were fairly satisfactory for investigations involving the entire drainage area, they, also, were inadequate for the intensive studies which would be required for the smaller tributaries. Consequently steps were at once taken, in cooperation with the U. S. Weather Bureau, to secure additional stations.

During the years 1913 and 1914 sixteen new stations were established as follows:

1. Rainfall stations, established by the U. S. Weather Bureau, at New Carlisle, Lake View, Versailles, St. Paris, Mt. Healthy, Fernbank, and Germantown.

2. Rainfall and river stations, established by the Dayton Flood Prevention Committee in cooperation with the U. S. Weather Bureau and maintained by the Weather Bureau, as follows:

Sidney—On the Miami River
Tadmor—On the Miami River
West Milton—On the Stillwater River
Springfield—On the Mad River

3. River stations established by the Dayton Flood Prevention Committee:

Germantown—On Twin Creek
Wright—On Mad River
Seven Mile—On Seven Mile Creek

Four Mile—On Four Mile Creek
Springfield—On Buck Creek

Since 1914 additional rainfall stations have been established by the U. S. Weather Bureau at Oxford, West Manchester, Eaton, Xenia, and Marysville.

A rainfall station was established by the Weather Bureau at Woodstock in August, 1916, but owing to the difficulty of securing satisfactory records the station was discontinued a few months later. The Dayton Flood Prevention Committee established a rainfall station at Moraine Park, about five miles south of Dayton, in March, 1915.

In the fall of 1915 the Miami Conservancy District, which had taken over the work of the Dayton Flood Prevention Committee, established a river station on Loramie Creek at Lockington. In the spring of 1916, after the retarding basin plan of flood control had been decided upon, the District established eleven new river stations as follows:

Fort Loramie—On Loramie Creek
Newport—On Loramie Creek
Troy—On Miami River
Tippecanoe City—On Miami River
Miamisburg—On Miami River
Franklin—On Miami River
Middletown—On Miami River
Pleasant Hill—On Stillwater River
Medway—On Mad River
Ingomar—On Twin Creek
Dayton—On Wolf Creek

The stations on Loramie and Twin Creeks, on the Stillwater and Mad Rivers, and the Troy and Tippecanoe City stations on the Miami River, were located within the proposed retarding basins. The primary purpose in establishing these was to secure records of flood heights, before the dams were started, for use in settling questions that may arise when construction work is finished. The stations on Wolf Creek and on the lower Miami River, at Miamisburg, Franklin, and Middletown, were established primarily for use in flood forecasting. A station on the Miami River at Elizabethtown, near the mouth of the river, was established for the same purpose in June, 1918. A station on the Miami at New Baltimore was established by the Local Weather Bureau Office at Cincinnati, January 14, 1916, for use in forecasting flood stages on the Ohio River.

FIG. 2.—GAGING STATION ON LORAMIE CREEK AT LOCKINGTON.

The U. S. Engineer Office, First Cincinnati District, established a river station on the Miami at Venice in June, 1915, and maintained records until July 1, 1920. At that time the Conservancy District established a gage and continued the records.

In the summer of 1918, after the construction of the flood prevention works had gotten under way, the Miami Conservancy District established river and rainfall stations at the Germantown, Englewood, Taylorsville, and Huffman Dams, a rainfall station at the Lockington Dam, and also installed rain gages at the Fort Loramie, Pleasant Hill, and Ingomar stations. A river gage was installed at Miller's Ford, just south of Dayton, in August, 1919, for use in determining discharges at Dayton during the progress of the river improvement work, since this work affected the rating curve for the Main Street section where the gage has always been located.

Figure 1, page 17, shows the gaging stations being maintained in September, 1919. Different symbols are used to distinguish between the rainfall, river and rainfall, and river stations, but no distinction is made between stations maintained by the U. S. Weather Bureau and those maintained by the Miami Conservancy District. Table 1 gives pertinent data relating to the various river stations, including the authorities maintaining the records. Figure 2 shows the station on Loramie Creek at Lockington.

A cable station, for use in measuring the larger floods, was installed by the District at Taylorsville, about two miles south of the Tadmor station, in May, 1916. A view of this station is shown in figure 3. The old Miami and Erie Canal crosses the valley on a fill at this place, thus causing the entire flow during large floods to be contracted from a width of about a half mile to a width of about 280 feet.

Automatic recording river gages, of the electric transmission type, were installed by the District at the Dayton and Hamilton stations in the spring of 1917. These gages are located in the offices of the District and consequently furnish accessible information regarding river stages at all times.

Four small plats, where the rainfall, runoff, and soil absorption could be measured, were established at Moraine Park, about five miles south of Dayton, in March, 1915. A standard rain gage was installed at the writer's residence in June, 1916; and a second gage was installed under the trees at the same place in July, 1919, in order to determine the amount of rainfall inter-

Table 1.—Data Relating to River Gaging Stations in the Miami Valley

Station	Stream	Location of Gage	Established	Elevation of Zero†	Flood Stage	Width at Average Low Water Feet	Width at Bank Full Stage Feet	Lowest Known Water Stage	Lowest Known Water Date	Highest Known Water Stage	Highest Known Water Date	Maintained by	Records	Drainage Area Square Miles
Sidney	Miami River	North St. Br.	2- 1-14	926.46	10	110	260	— 1.5	9-19-17	17.9	3-25-13	U.S.W.B.	daily	555
			* 9-19-19	924.65	12									
Piqua	" "	N. Main St. Br.	8- 4-05	.....	14	300	350	0	1-16-10	23.3	3-25-13	"	⊙ "	842
			* 5- 1-10	849.10	12									
Troy	" "	Broadford Br.	4- 5-16	810.74	9	165	265	0.5	7-30-19	14.2	3-25-13	M.C.D.	"	908
			* 7- 9-16	809.24										
TippecanoeCity	" "	Carlisle Pike Br. 1 mi. E. of town	3-20-16	783.7	8	100	250	0.2	2- 8-18	15.	3-25-13	"	"	1016
Tadmor	" "	National Pike Br.	12-22-13	763.68	12	160	250	1.7	8-21-17	25.4	3-25-13	U.S.W.B.	"	1128
Taylorsville	" "	R. R. trestle above dam	7-29-18	700.	70	340	420	63.6	8-22-18	81.7	3-25-13	M.C.D.	"	1133
Dayton	" "	Main St. Br.	10- 1-92	723.73	18	350	600	0	10-27-95	29.0	3-26-13	U.S.W.B.	"	2525
Miamisburg	" "	Linden Ave. Br.	3-14-16	681.09	....	350	665	0.3	8-14-19	31.0	3-26-13	M.C.D.	"	2720
			* 9- 5-16	680.09	17									
Franklin	" "	Suspension Br.	3-15-16	658.41	17	260	365	0.7	10-18-17	23.0	3-26-13	"	"	2785
Middletown	" "	Third St. Br.	6-22-16	624.65	17	250	300	1.0	7-11-18	29.0	3-26-13	"	"	3162
Hamilton	" "	High St. Br.	11-16-04	564.56	12	450	500	0.1	8-27-19	38.5	3-26-13	"	§ "	3672
Venice	" "	Highway Br. ½ mi. S. E. of town	6-14-15	520.22	16	265	355	1.2	11- 6-17	38.0	3-26-13	WarDep.	"	3800
			* 6-28-20	518.42	18							M.C.D.	"	
New Baltimore	" "	Highway Br.	1-14-16	504.0	25	340	485	.....	.....	38.0	3-26-13	U.S.W.B.	flood	3830
Elizabethtown	" "	Highway Br. ½ mi. S. E. of town	6-12-18	453.33	9	300	700	— 0.6	6-14-18	38.6	2-14-84	M.C.D.	daily	5430
Dayton	" "	Highway Br. at Miller's Ford	9- 7-19	712.86	15	300	470	0.2	9-13-19	27.	3-26-13	"	"	2600
Fort Loramie	Loramie Creek	Highway Br. W. side of town	4-14-16	933.05	6	18	122	0.7	8-14-16	16.0	3-25-13	"	"	85

*Reset. †Mean Sea Level Datum. ⊙Flood Stages 1-10-07 to 1-11-11. §Flood Stages 11-16-04 to 2-28-10. *New Gage.

Table 1.—*Continued*

Station	Stream	Location of Gage	Established	Elevation of Zero†	Flood Stage	Width at Average Low Water Feet	Width at Bank Full Stage Feet	Lowest Known Water Stage	Lowest Known Water Date	Highest Known Water Stage	Highest Known Water Date	Maintained by	Records	Drainage Area Square Miles
Newport.....	Loramie Creek	Highway Br. ¼ mi. S. of town.....	4-13-16	923.90	8	15	110	0	8-15-19	15.0	3-25-13	M.C.D.	daily	157
Lockington...	" "	Highway Br. ½ mi. W. of town.....	9-13-15	875.99	7	112	112	0.8	8-29-18	15.6	3-25-13	"	"	255
Pleasant Hill..	Stillwater River	Highway Br. ½ mi. N. W. of town..	4- 7-16	846.55	13	200	249	1.2	1- 1-17	17.5	3-25-13	"	"	453
West Milton..	" "	C.C.C.&St.L.R.R. Br. near Troy Pk.	1- 3-14	812.97	7	110	200	0.5	8-27-18	28.0	3-25-13	U.S.W.B.	"	600
Englewood....	" "	R. R. trestle above dam.....	5- -18	0.	780	190	220	773.	7- 6-19	793.8	3-25-13	M.C.D.	"	651
Springfield...	Buck Creek	Plum St. Br.....	7-13-14	908.20	7	70	90	0.3	12-20-17	12.3	3-25-13	"	"	163
Springfield....	Mad River	Old Mill S. of C.C. C.&St.L. Br.#121	2- 5-14	887.81	10	50	100	1.0	8-22-18	19.2	3-25-13	U.S.W.B.	"	488
Medway.....	" "	Highway Br. 1 mi. S. E. of Medway	3- 7-16	837.16	8	85	145	0.3	12-17-17	....	3-25-13	M.C.D.	"	579
Wright.....	" "	Covered Highway Br. at Simms...	11-19-14	783.91	7	70	200	0.8	9-15-19	14.0	3-25-13	"	"	652
Huffman.....	" "	200 ft. above R. R. trestle above dam	6- 6-18	780.	6	120	145	1.4	8-24-18	12.5	3-25-13	"	"	671
Dayton.....	Wolf Creek	First Highway Br. above city.....	6-12-16	743.76	12	30	75	.....	.....	14.5	3-25-13	"	flood	71
Ingomar.....	Twin Creek	Highway Br. 1½ mi. W. of town..	4-12-16	815.42	12	115	257	0	9-16-19	28.0	3-25-13	"	daily	193
Germant'n Dam	" "	Sump above dam	6- 1-19	720.	10	150	175	3.0	11- 4-13	26.0	3-25-13	"	"	270
Germantown..	" "	Highway Br. 1 mi. W. of town.....	4- 9-14	714.0	7	100	200	0.6	10- 4-19	17.0	3-25-13	"	"	272
			*8-14-14	712.73	8					18.3				
Seven Mile...	Seven Mile Creek	Covered Br. ¼ mi. W. of town.....	11-16-14	623.10	10	25	200	1.1	11-16-14	17.0	3-25-13	"	"	128
Four Mile....	Four Mile Creek	H. Br. Eaton Rd. N.W. of Hamilton	11-17-14	618.69	7	15	100	0.9	8-25-18	18.0	3-25-13	"	"	178

*Reset. †Mean Sea Level Datum.

cepted by trees. Plats similar to those at Moraine Park were established at the Taylorsville Dam in July, 1920, so that rainfall and runoff experiments could be carried on, using a sprinkling can to reproduce rainfall effects.

RECORDS BEING SECURED

Daily rainfall records, including notes regarding the times of beginning and ending of the rain, are being secured at all but one or two of the rainfall stations shown in figure 1. In addition, automatic graphical records of rainfall and river stage are being secured at Dayton. Similar automatic rainfall records are, of course, being taken at several regular Weather Bureau stations surrounding the Miami Valley, such as Cincinnati, Indianapolis, Fort Wayne, Columbus, and Toledo. Daily records of gage heights are being secured at all river stations except New Baltimore and the one on Wolf Creek, where only flood records are being secured.

The daily river and rainfall observations taken at the Weather Bureau stations are recorded on the usual forms and are reported at the end of each month. Observers at some of the stations shown on figure 1 send their reports to the local Weather Bureau office at Dayton. The others report to the office at Columbus. Observations taken at the Miami Conservancy District's stations are recorded on postal card forms and are mailed to the headquarter's office at Dayton at the end of each week.

Special highwater readings of the river gages, for use in determining flood hydrographs, are being secured by the District at the following stations:

Sidney—On the Miami River
Piqua—On the Miami River
Tadmor—On the Miami River
Miamisburg—On the Miami River
Franklin—On the Miami River
Middletown—On the Miami River
Lockington—On Loramie Creek
West Milton—On Stillwater River
Springfield—On Buck Creek
Springfield—On Mad River
Wright—On Mad River
Dayton—On Wolf Creek
Germantown—On Twin Creek
Seven Mile—On Seven Mile Creek
Four Mile—On Four Mile Creek

FIG. 3.—CABLE GAGING STATION ON MIAMI RIVER AT TADMOR.

During the flood of March, 1913, the entire flow of the Miami River at Tadmor passed between the abutments shown in the picture, destroying the bridge superstructure and the two piers.

In general, these readings are taken every hour during the rising flood, every two hours during the day following the time of maximum stage, and then three times each day until the water has fallen to about the stage existing before the rise began. Each day's readings are recorded on a special postal card form and mailed to the headquarter's office as soon as possible.

Special reports from the greater number of both river and rainfall stations, for use in forecasting flood heights, are made direct to the Conservancy District during critical periods, as well as to the Weather Bureau. These reports are made by telephone or telegraph as soon as the rainfall amounts to 0.70 of an inch, provided it has fallen in 24 hours or less; or whenever there is a sudden rise in the river stage amounting to three feet or more. A confirmation of each report is made by mail as soon as the message has been telegraphed or telephoned. These reports make possible the accurate forecasting of flood conditions and also furnish valuable information regarding flood runoff and storm rainfall.

Rainfall measurements are recorded to the nearest hundredth of an inch. Where the precipitation is less than a hundredth of an inch the amount is indicated by a capital "T" meaning "trace." River gage readings are observed and recorded to the nearest tenth of a foot at all stations except Venice. At Venice, where the gage is of the Mott type, the observations are taken to the nearest hundredth of a foot.

Readings to hundredths of a foot may be practicable at times during ordinary and low water stages where the stations are equipped with chain and weight or Mott gages, or with vertical staff gages graduated to hundredths. However, where the gages are of the vertical staff type, graduated to tenths only, it is doubtful if such precision is ever warranted, especially where the observers have had no technical training, as is generally the case. During flood conditions, or if there is a strong wind blowing, the water will rise and fall, intermittently, from a tenth to a half a foot or more; so that readings to hundredths, while mechanically possible with certain gages, are accurate only to tenths of a foot at the best.

GAGES IN USE

Standard U. S. Weather Bureau rain and snow gages are being used at all rainfall stations. The regular Weather Bureau station at Dayton is also equipped with tipping bucket gage.

A chain and weight river gage is in use at the New Baltimore station, and Mott tape gages at Piqua and Venice. The other river stations are equipped with vertical staff gages. Automatic recording river gages of the electric transmission type are in use at Hamilton and Dayton.

The engineers of the Miami Conservancy District prefer the vertical staff gage to any other type, leaving out of consideration the sloping gages which are so expensive that they are feasible only in exceptional cases. The principal objection to the chain and weight gage is that the chain gradually stretches, thus requiring the continual checking of the chain length and the correcting of the observer's reports. Another objection, which applies also to the Mott gage, is that the boxes, having a somewhat mysterious appearance, are frequently broken into and the gages damaged.

DISCHARGE MEASUREMENTS

Measurements of discharge are made by the District at all river stations in the valley except New Baltimore and the Germantown, Englewood, Taylorsville, and Huffman dams. Measurements are not made at these places since they are close to the other stations and since the conditions due to the construction work are unfavorable for the securing of accurate data. The station at Venice was well rated by the engineers of the War Department, First Cincinnati District, during the flood of July, 1915.

Measurements are made during flood periods and more or less periodically during normal or low water conditions. They serve to determine the relations between gage heights and discharge, thus enabling the calculation of station rating tables and the compilation of daily stream flow records. Moreover, the inspections by the hydrographers furnish checks on the accuracy of the observer's readings and also supply information regarding channel conditions, effects of vegetation on stages, and the like.

Periodic measurements of discharge are also made on various artificial channels, carrying water for industrial use, as follows:

Miami and Erie Canal north of Fort Loramie
Miami and Erie Canal Feeder at Sidney
Tail Race at Slusser-McLean Company's Plant at Sidney
Miami and Erie Canal at Lockington
Miami and Erie Canal Feeder north of Lockington

Miami and Erie Canal at Piqua
Tail Race at Waterworks Pumping Plant at Piqua
Miami and Erie Canal at Troy
Mill Race at Troy
Miami and Erie Canal at Tippecanoe City
Head Race at Tranchant & Finnell Mills at Osborn
Miami and Erie Canal Feeder at Findlay Street, Dayton
Miami and Erie Canal Feeder Wasteway below Findlay Street, Dayton
Miami and Erie Canal at Warren Street, Dayton
Dayton Hydraulic Company's Canal at Findlay Street, Dayton
Miami and Erie Canal at West Carrollton
Hydraulic Canal at West Carrollton
Miami and Erie Canal at Miamisburg
Tail Race at Grove & Weber Co.'s Plant, Miamisburg
Tail Race at Ohio Paper Co.'s Plant, Miamisburg
Tail Race at Miamisburg Paper Co.'s Plant, Miamisburg
Miami and Erie Canal at Franklin
Hydraulic Canal at Franklin
Miami and Erie Canal at Middletown
Hydraulic Canal at Middletown
Miami and Erie Canal at Hamilton
Hydraulic Canal above Reservoir at Hamilton
Hydraulic Canal at Niles Tool Works, Hamilton
Old River at Hamilton
Head Race at Bentel Margedant Plant, Hamilton
Wasteway at Ohio Electric Power Plant, Hamilton

These measurements furnish the information needed in benefit and damage assessments as well as in design of local channel improvements. They also furnish the additional data needed in calculating total runoff at river stations. Gagings at some of the above sections have recently been discontinued due to changes made in connection with the construction work.

All gagings are made with the small Price current meter, combination type, using the penta commutator whenever the velocities are so high that single revolutions of the meter cannot be accurately counted. Observations are taken by the two-point method whenever feasible. During low water conditions measurements are made by wading, using the six-tenths depth method if the water is less than two feet deep. During floods it is frequently necessary to resort to the surface method. In such cases coefficients of from 0.8 to 0.9 are used to reduce the surface

velocities to mean velocities, the particular coefficient used in a given case being determined from a study of vertical velocity curves taken at the given station. Stay lines have been used in some cases.

The two-point method of measurement has been tested by about fifty vertical velocity curves, taken at various locations among the fifty odd gaging stations, in artificial as well as natural channels. The average of the ratios of the velocity by the two-point method to the velocity determined from the curve was found to be 0.994; the average error of the two-point method, obtained by averaging, arithmetically, the differences between the ratios and unity, was 1.24 per cent; and the maximum error for a single curve was 7.5 per cent.

Gage readings are taken before and after each measurement, to the nearest half tenth of a foot wherever practicable. Soundings are recorded to the nearest tenth. Observations are taken in at least ten but not more than twenty vertical sections during each gaging, regardless of the width of the stream. If the velocity varies greatly across the stream the sections are spaced closer together than usual.

In computing discharges from field notes the velocities and depths measured in a given vertical are assumed to represent average conditions in a width of channel extending, on each side, half way to the adjacent verticals. This method has been found to give fully as satisfactory results as the method of averaging velocities and depths in adjacent vertical sections to get the average conditions in the width of channel between sections. The latter method requires two operations not necessary in the former.

Current meters are rated at least once each year, more frequently if necessary. However, the experience of the Miami Conservancy District has been that the ratings of individual instruments, where the instruments have received proper care, seldom differ more than one or two per cent from the composite table furnished by the manufacturers.

The meters were formerly rated by the Bureau of Standards at Washington. Recently, however, they have been rated in the river at Dayton, at a location just above an old concrete dam, where still water exists. Ratings are made with the meters suspended by cables and held in place by lead torpedo weights, the conditions being made as nearly as possible like those under which the meters are used.

STREAM FLOW RECORDS

Daily stream flow records are being compiled for the following stations:

Sidney—On the Miami River
Piqua—On the Miami River
Tadmor—On the Miami River
Dayton—On the Miami River
Franklin—On the Miami River
Hamilton—On the Miami River
Venice—On the Miami River
Lockington—On Loramie Creek
Pleasant Hill—On Stillwater River
West Milton—On Stillwater River
Springfield—On Buck Creek
Springfield—On Mad River
Wright—On Mad River
Germantown—On Twin Creek
Seven Mile—On Seven Mile Creek
Four Mile—On Four Mile Creek

The records are tabulated on forms similar to those used by the U. S. Geological Survey, one sheet being used for each year at each station. These sheets give the daily stages and discharges, the mean monthly discharges in second feet and in second feet per square mile, the monthly runoff in inches depth over the drainage area and in acre feet, the maximum and minimum discharges for each month, the total runoff for the year, and the mean, maximum, and minimum rates of runoff for the year.

The records are believed to be as accurate as it is feasible to determine such data on streams similar to those in the Miami Valley. They are, of course, more accurate for the larger streams having the flatter slopes than for the smaller streams having the steeper slopes. The records for the Four Mile Creek station are more unsatisfactory than those for any other station in the valley, due to the shifting of the control during floods. This shifting occurs during small rises of two or three feet as well as during the larger floods, owing to the sand and gravel deposits at the station and to the steep slope of the stream, about fifteen feet per mile.

PUBLICATION OF DATA

The daily rainfall records at Weather Bureau stations are published by the U. S. Weather Bureau in their "Climatological

Data." The records at the Miami Conservancy District's stations are not being published.

Daily gage heights at the river stations maintained by the U. S. Weather Bureau are published annually in their "Daily River Stages at River Gage Stations on the Principal Rivers of the United States." Summaries of discharge measurements, daily stream flow records, and descriptions of stations, for stations where stream flow records are being compiled, are published by the U. S. Geological Survey in their water supply papers. Records secured at the other river stations are not published. Discharge measurements made on artificial channels are not published except where the results are needed to determine total runoff at river stations. In such cases the results are published in the U. S. Geological Survey water supply papers.

The data on rainfall, runoff, and soil moisture collected at Moraine Park is given in full in chapter III of this volume. The data on rainfall intercepted by trees is given in chapter VII.

River or rainfall records secured by the Miami Conservancy District and not published may be obtained from the District at the cost of blue printing.

CHAPTER III.—MORAINE PARK EXPERIMENTS

In February, 1915, it was decided to make a series of field investigations of precipitation, surface runoff, and soil moisture at isolated plats of various characteristics, the object being to obtain data on the conditions under which surface, or flood, runoff takes place. For this purpose four small experimental plats were located in an orchard at Moraine Park, the home of Colonel E. A. Deeds, about five miles south of Dayton. It was recognized, of course, that these plats were too small and too few in number to be representative of the average conditions throughout the Miami Valley. In fact, it was known that the conditions are not typical. The area just south of Dayton consists of deep glacial deposits of sand and gravel, covered with a thin layer of surface soil, in the form of comparatively steep eskers and moraines; while the areas north of Dayton are slightly rolling glaciated areas with deeper surface soil underlaid by materials of various nature and geological age. However, it was thought that if a detailed study could be given to the rainfall and runoff conditions at selected places, by experienced observers, valuable information regarding the laws of runoff could be secured.

For a study of the laws of runoff and the relation of runoff to rainfall small experimental plats possess certain definite advantages over the much larger drainage areas which exist above the stream gaging stations. For instance the slopes of the ground surface within the plats, as well as the character of the soil and surface covering, can be accurately determined without making elaborate and costly surveys. In fact, the plats may be located so that definite comparisons can be secured between the runoff from areas having different surface conditions. Certain questionable features pertaining to the larger areas are practically eliminated in the smaller, such as the absorption of runoff by the soil before it reaches the drains and the amount of runoff contributed by the ground water storage.

While the Moraine Park experiments do not furnish conclusive evidence on all phases of the subject, it is believed that the results are worthy of presentation.

DESCRIPTION OF PLATS

Four plats, each five feet square, are located in open places in the orchard, two on level ground and two on a hillside, the two sets being about a hundred feet apart, and the two plats of each set being about ten feet apart. A standard rain gage was installed near each set. The plats on the hillside were placed where the slope of the ground is about eighteen feet per hundred. One plat on the hillside and one on the level ground were located where the surface covering is a heavy blue grass sod. The other two were located where the sod had been removed leaving the soil bare.

The upper two feet of soil on the hillside is a yellow, sandy loam containing some clay and gravel; the upper two feet where the level plats were established is a similar material except that it contains a larger proportion of gravel. At both places the upper foot contains considerable humus. Of course the soil under the blue grass covering is practically full of roots, some of which extend to depths of 2 feet or more. The material underlying the 2-foot layer of loam, in both cases, is a mixed sand and gravel of glacial origin. On the hillside there is a fairly definite division between the loam and the underlying deposits. On the level ground the proportion of sand and gravel increases more or less uniformly with the depth below the surface until a depth of about two feet is reached. Below this depth the amount of silt and clay present is negligible.

Mechanical analyses of typical samples of the surface soil taken on the level and on the hillside showed that the proportion retained on a quarter inch sieve is about 30 per cent, by weight, for the former and about 7.5 per cent for the latter. The analyses of the portions passing the quarter inch sieve, made by the Bureau of Soils, U. S. Department of Agriculture, gave the following results:

	Percentage by Weight	
	Level	Hillside
Fine gravel, 2 to 1 mm.	3.0	2.6
Coarse sand, 1 to 0.5 mm.	11.8	9.8
Medium sand, 0.5 to 0.25 mm.	12.2	9.0
Fine sand, 0.25 to 0.10 mm.	29.6	28.0
Very fine sand, 0.10 to 0.05 mm.	7.4	8.7
Silt, 0.05 to 0.005 mm.	19.6	24.0
Clay, less than 0.005 mm.	16.4	18.0

The plats were isolated from the adjacent ground by corrugated iron strips set into the ground about eight inches and ex-

tending above the ground about four inches. In setting these strips care was taken not to disturb the ground inside the plats. Concrete was placed around the outside of the corners so as to prevent leakage at the joints. Of course it is quite possible that some water may creep down the inside edges of the iron strips thus slightly increasing the soil percolation.

A galvanized iron tank, eighteen inches in diameter and four feet deep, to catch the surface runoff, was set in the ground just outside the lower corner of each plat, and was connected with the inside of the plat by a joint of three-inch sewer pipe, laid in concrete. A wire screen, to keep out vermin, was fastened over the upper end of each sewer pipe. The tanks were tested and found to be water-tight before being installed; and were tested at intervals after installation, no leaks being found at any time. They were provided with suitable tight fitting covers so that no water except surface runoff from the plats could be caught, and so that the evaporation within the tanks would be reduced as much as possible. The capacity of each tank is equivalent to a runoff of about 3.0 inches depth over the plat with which it is connected.

Some trouble was encountered at times due to leaves stopping up the screens and causing the runoff to spill over the tops of the iron strips. This occurred mostly at the plat on the hillside having the bare soil surface. The screens were later replaced by wire mesh having openings about three-eighths of an inch square, after which more satisfactory records were obtained.

The work of establishing the plats and installing the gages was completed March 4, 1915, and the measurements were begun the following day.

METHODS OF MEASUREMENT

Measurements of rainfall, runoff, and soil moisture have been made more or less regularly since the plats were established. The endeavor has been to secure observations just before and just after each rain, and also to secure measurements of soil moisture once or twice a week between rains, to determine the rates of drying of the soil. Owing to the pressure of other work it has not always been possible to adhere strictly to the above plans. Observations were also discontinued for short intervals during the winter months, as, for instance, during the severe winter of 1917 and 1918. During the first year the soil moisture determinations were made rather irregularly. Some-

times samples were taken to depths of 18 or 24 inches, but more frequently they were only taken to depths of 12 inches. Since March, 1916, however, samples have been taken systematically to depths of 24 inches.

Where precipitation occurred on two or more days between successive readings of the gages it is possible to estimate the daily amounts at Moraine Park from the daily records taken by the U. S. Weather Bureau at Dayton. While such estimates may be considerably in error during summer thunder-storms it is not believed that they are greatly in error at other times. Valuable information regarding intensities of rainfall is also furnished by the graphical automatic records being secured at the Dayton Weather Bureau station. Notes regarding rainfall and runoff conditions were made by the writer at his home in Carrmonte, about two miles north of Moraine Park.

Rainfall measurements were made in the usual manner, that is, using the regular rain gage measuring sticks. The amounts of runoff were determined by measuring distances from the tops of the cans down to the water surfaces, using yard sticks, and reading distances to eighths of an inch. A depth in the can of an eighth of an inch corresponds to a depth over the plat of about 0.009 of an inch.

The amount of moisture in the soil, under the sod and under the bare surface, was determined by taking samples, weighing them, drying, and reweighing. Samples weighing about a kilogram, or about two pounds, were taken at intervals of about six inches in depth down to a depth of about two feet. During the first few months samples were taken close to the plats on the hillside as well as close to those on the level ground. Slightly different results under similar surface coverings were obtained at the two places, the moisture content of the soil on the hillside generally being a little greater than that of the soil on the level ground. This was probably due to the much larger proportion of gravel in the soil at the latter place. Later on samples were taken from beneath the sod and bare soil surfaces at a place on level ground, from 25 to 100 feet southeast of the level plats, where the soil was very similar to that on the hillside.

The samples were placed in paper sacks and dried in the furnace room at Moraine Park. Harvard scales, reading to tenths of a gram, were used in weighing. Weights were tested and adjusted using standard scales of known accuracy. The samples were dried and reweighed until their dry weight became

constant, before they were discarded. When the investigations were begun it was attempted to dry the samples by leaving them in small incubators which could be kept at a constant temperature of about 100° Fahrenheit. It was found, however, that owing to poor air circulation in the incubators it required several weeks to dry the samples thoroughly. They were then placed in the furnace room, directly over the furnace, where they dried out in a few days; or, when the furnace was not being used, they were placed on shelves above a small coal water heater, which was used every morning.

Determinations of the weight per cubic foot of the upper two feet of soil were made December 8, 1919. Samples were taken by boring down with a post hole auger, and were weighed, dried and reweighed in the laboratory at the headquarters office. The cubical contents of the samples were obtained by weighing the amounts of dry sand of known density required to fill the holes from which the samples had been taken.

RESULTS OF OBSERVATIONS

Weight of Soil per Cubic Foot

The data on the weight per cubic foot of the upper two feet of soil is given in table 2. Samples 1 and 4 were taken from beneath the bare surface on the level ground, where the samples for determining the moisture content of the soil have been taken regularly. Samples 2 and 3 were taken from beneath the sod surface on the level ground. Sample 5 was taken from beneath the sod surface near the plats on the hillside.

Table 2.—Determinations of Weight per Cubic Foot of Moraine Park Loam

Sample Number	Volume of Sample	Weight when taken	Weight of Moisture in Sample	Dry Weight of Sample	Weight per cubic foot when taken	Weight per cubic foot when dry	Moisture in Sample
	Cubic feet	Pounds	Pounds	Pounds	Pounds	Pounds	Per cent*
1	0.410	47.1	9.9	37.2	115	91	26.6
2	0.652	81.4	15.4	66.0	125	101	23.3
3	0.710	85.9	13.6	72.3	121	102	18.8
4	0.510	58.6	8.8	49.8	115	98	17.8
5	0.713	86.3	13.1	73.2	121	103	17.9
Average..	.....	.....	.....	.....	119.4	99.0	20.9

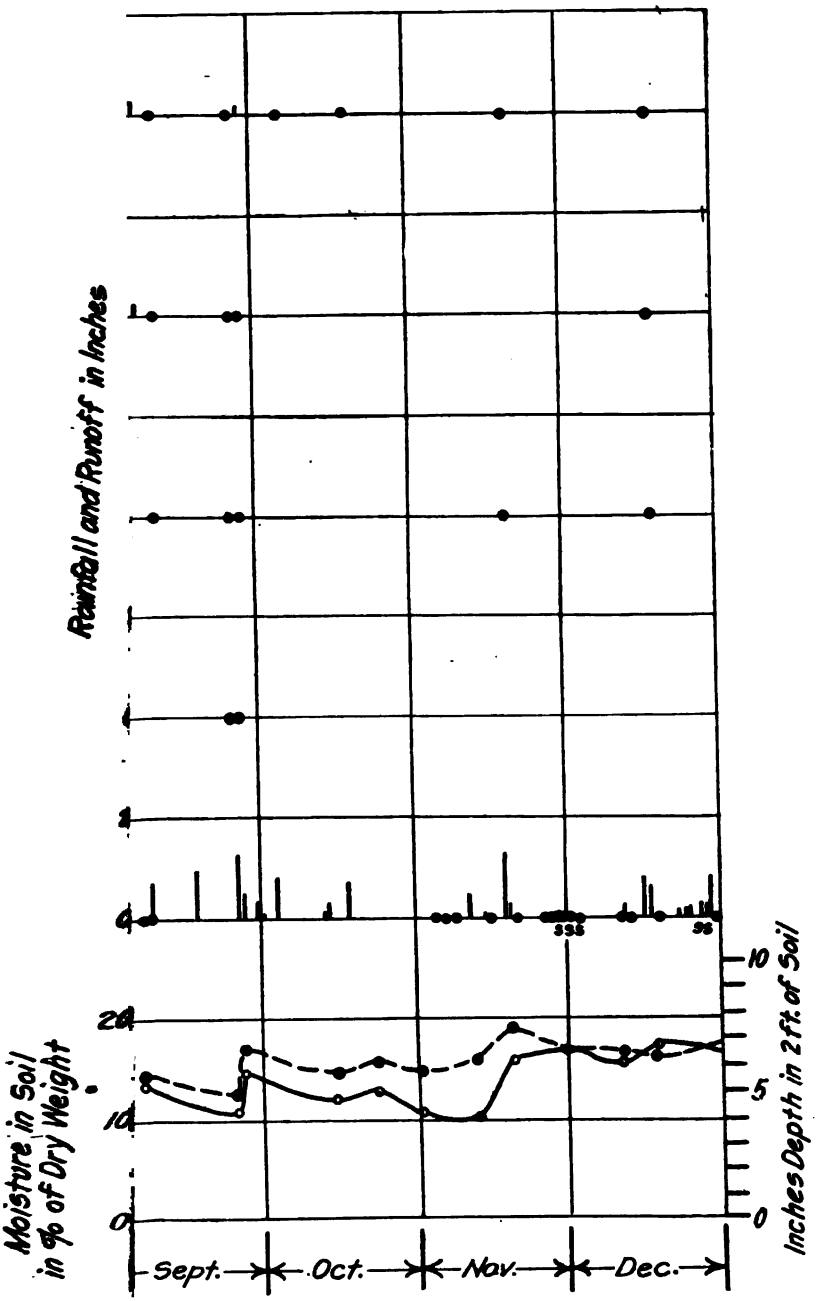
*Based on dry weight.

It will be noticed that there was about 20.9 per cent of moisture in the soil at the time the samples were taken; that the weight per cubic foot of the soil when taken varied from 115 to 125 pounds, averaging 119.4 pounds; and that the weight per cubic foot when dry varied from 91 to 103 pounds, averaging 99.0 pounds. In order to simplify the calculations an average value of 100 pounds has been used for the dry weight in the studies taken up later.

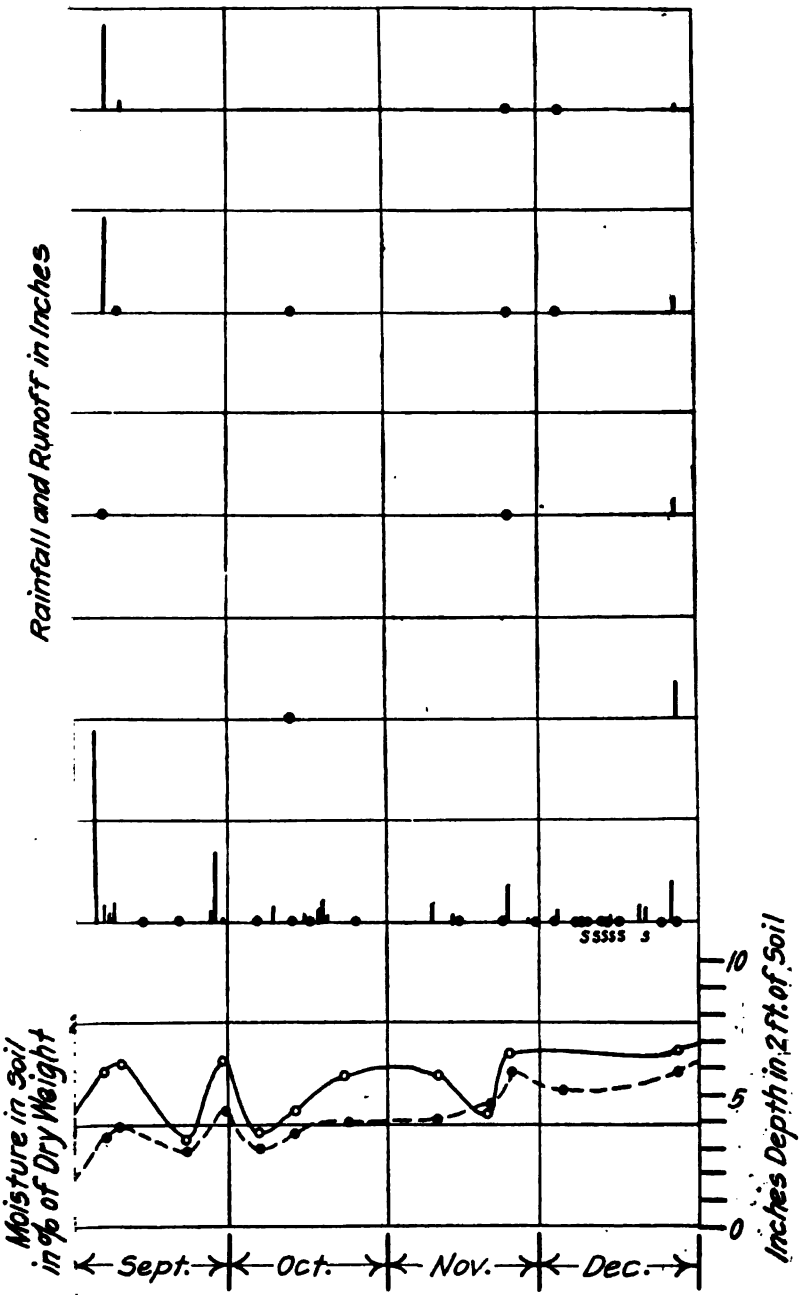
When the samples were taken there was, on the average, about 20.4 pounds or .33 of a cubic foot of water in each cubic foot of soil. While the upper two feet of soil at that time was about as wet as it ever gets under field conditions, the actual volume of the voids was probably a little greater than this. If a value of 2.7 is assumed for the specific gravity of the soil particles, an average value based on several laboratory determinations, the weight of a cubic foot of soil particles would be 169 pounds, the volume of the particles in one cubic foot of soil in place would be 0.59 of a cubic foot and the volume of the voids in one cubic foot would be 0.41 of a cubic foot. Consequently the maximum amount of moisture that could be present in the soil would be 41 per cent by volume or about 25.6 per cent of the dry weight.

Rainfall, Runoff, and Soil Moisture

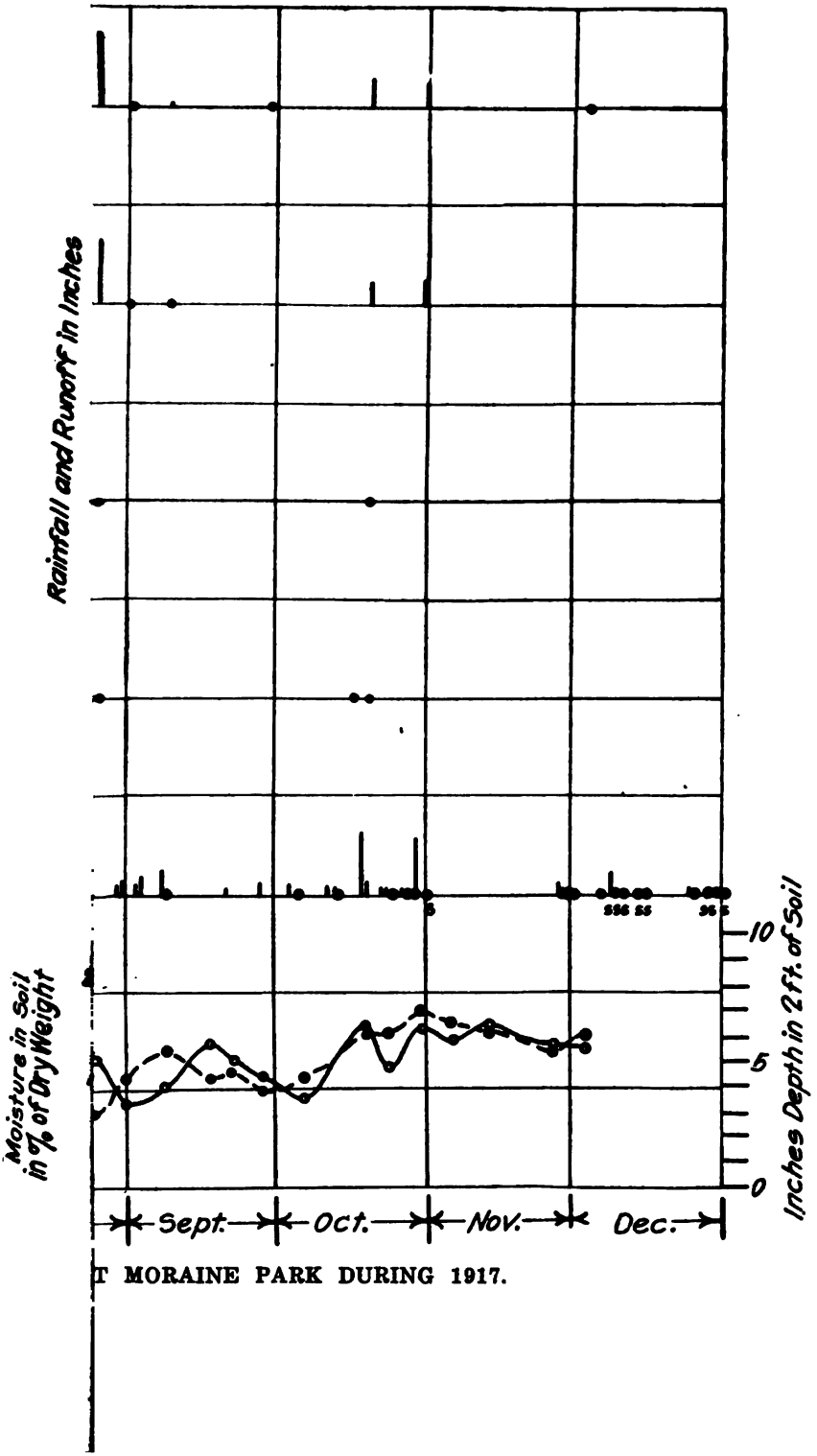
Table 3 gives the results of all observations of rainfall, runoff, and soil moisture, taken from the time the plats were established up to the end of October, 1919, about four years and eight months in all. Column 1 gives the date of observations. Columns 2 to 5, inclusive, give the moisture content of the soil under the sod covering, expressed as percentages of the dry weight. Column 6 gives the average moisture content of the soil under the sod covering calculated from the data in columns 2 to 5. Columns 7 to 11, inclusive, give corresponding data for the soil under the bare surface. Column 12 gives the observed rainfall. Columns 13 to 15, inclusive, give the data on runoff from the plats having the sod covering. Column 13 gives the runoff from the level plat, column 14 gives the runoff from the plat on the hillside, and column 15 gives the average runoff from the two. Columns 16 to 18, inclusive, give similar runoff data for the plats having the bare soil surface. The maximum and minimum records of soil moisture are set in bold face type so they can be easily located.



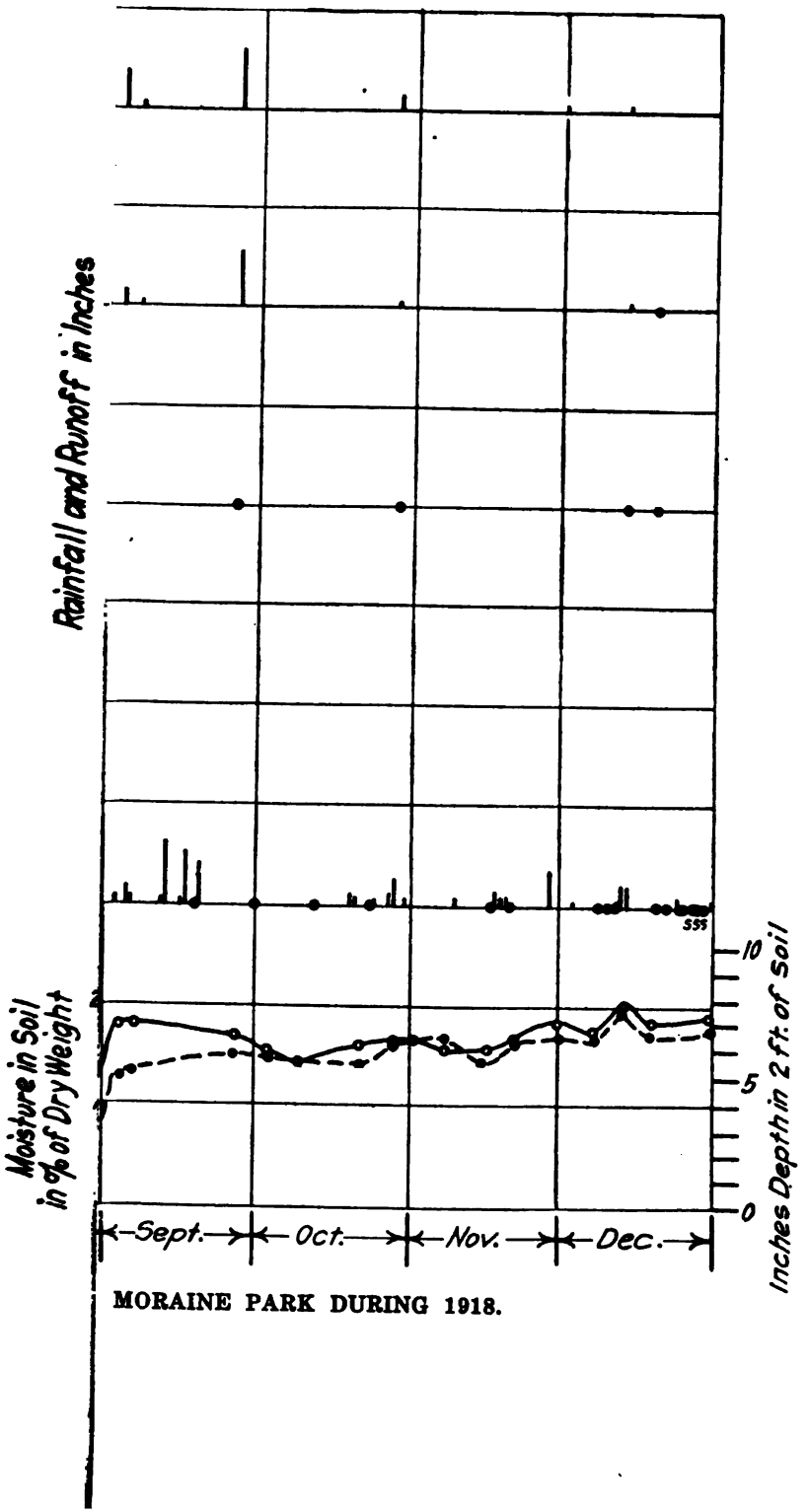
ORAINE PARK DURING 1915.



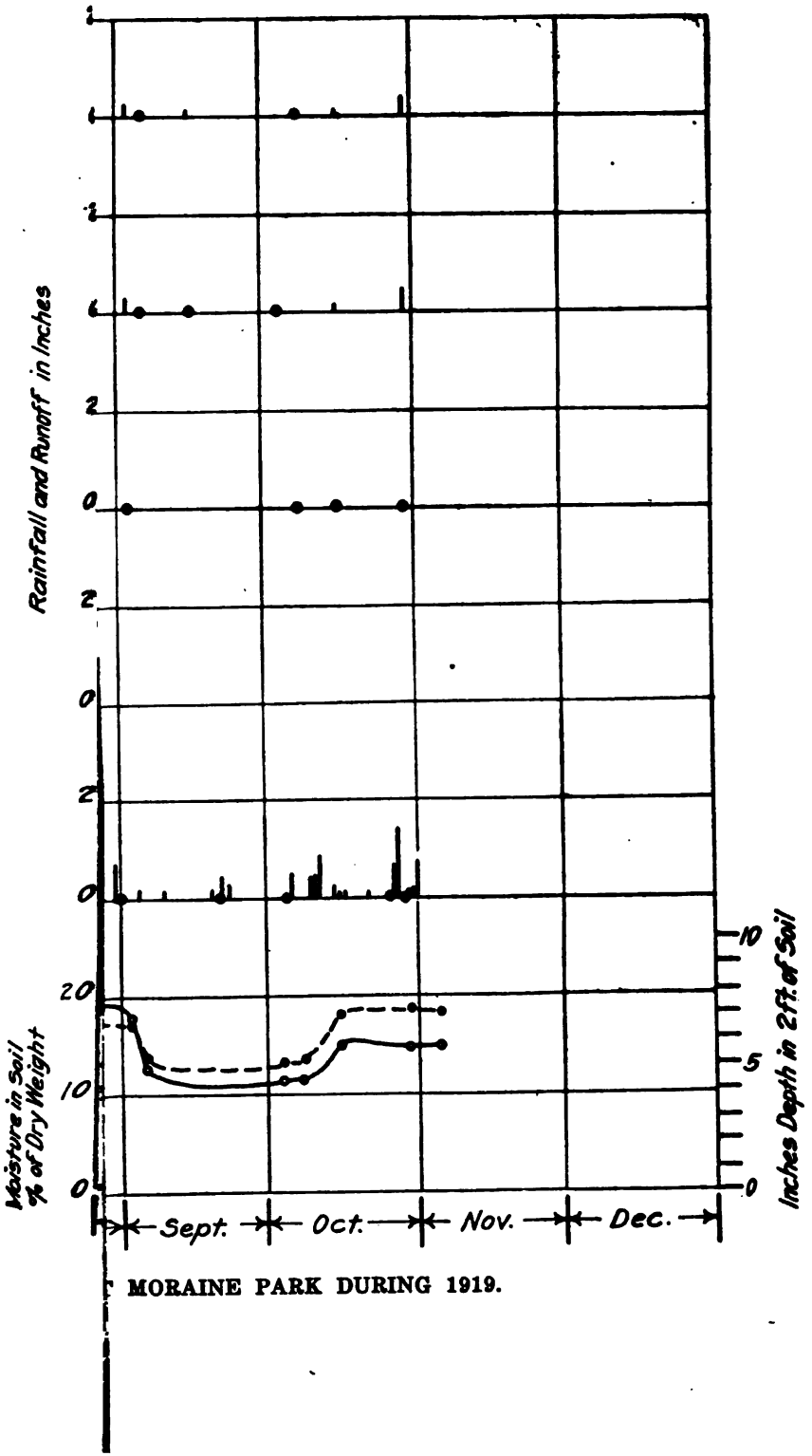
MORAINE PARK DURING 1916.



T MORAINE PARK DURING 1917.



MORAINE PARK DURING 1918.



MORAINE PARK DURING 1919.

The rainfall records given in column 12 were obtained by averaging the observations at the two rain gages, it being assumed that the differences in the two readings were due to instrumental and observational errors rather than to actual differences in precipitation at the two places. The differences were small in all cases, seldom exceeding 0.05 of an inch, except in cases where the gages had not been read for some time or where the precipitation had occurred as snow.

The data given in table 3 is shown graphically in figures 4 to 8, inclusive, each calendar year's records being shown on a separate sheet. The rainfall and runoff records, in inches depth, are platted as vertical bars in the upper parts of the diagrams, whenever the depth amounted to or exceeded a tenth of an inch. The occurrences of amounts less than a tenth of an inch are indicated by small circles. In platting the rainfall the amounts given in column 12 have been distributed over the days on which the precipitation occurred, as shown by the Dayton U. S. Weather Bureau records. This distribution was made on the assumption that the ratio of the rainfall on a given day to the total precipitation for a period including the given day was the same at both locations. While this assumption may be considerably in error during summer thunder-showers, as previously mentioned, it probably is not seriously incorrect during the more steady rains of the winter and spring. At any rate it gives the reader an idea of the general distribution of the rainfall and of the dates on which the greater part of the runoff occurred. Whenever the precipitation occurred in the form of snow a small "s" has been placed below the circle or line representing the precipitation.

Runoff records for each plat are shown, but the averages for the different types of surface covering, given in columns 15 and 18 of table 3, are not platted. The actual runoff records, given in table 3, are platted on the dates on which the observations were made. No attempt has been made to distribute the amounts or to plat them on the days on which they must have occurred.

The averages of the soil moisture determinations for the different types of surface covering, given in columns 6 and 11 of table 3, are shown in the lower parts of the diagrams. The percentages are platted as points, and the points are connected by lines. The points representing the moisture under the sod are connected by continuous lines, and those representing the

Table 3.—Results of Rainfall, Runoff, and Soil Absorption Experiments at Moraine Park

Date	Moisture in Soil, in per cent of dry weight										Rainfall in Inches	Runoff, in inches depth					
	Under Sod Cover, depth of					Bare Soil, depth of						Sod			Bare Soil		
	6″	12″	18″	24″	Average	6″	12″	18″	24″	Average		Level	Sloping	Average	Level	Sloping	Average
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
1915												Gages Established					
3- 4	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....						
3- 5	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.95	0	0	0	0	0	0
3- 6	.....	*12.8	.....	.....	12.8	.....	.....	.....	.....	.....	.06	0	0	0	0	0	0
3- 8	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	T	0	0	0	0	0	0
3-12	14.6	15.0	18.0	23.0	17.6	.....	.....	.....	.....	.....	0	0	0	0	0	0	0
3-16	*16.0	*15.6	*23.7	*24.9	15.8	.....	.....	.....	.....	.....	.03	0	0	0	0	0	0
3-19	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.27	0	0	0	0	0	0
3-20	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	T	0	0	0	0	0	0
3-22	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	T	0	0	0	0	0	0
3-25	16.6	18.0	21.2	23.6	19.8	15.0	15.8	19.3	17.6	16.9	.01	0	0	0	0	0	0
4-12	15.3	17.0	18.5	20.5	17.8	14.7	14.2	20.7	.....	16.5	1.05	0	.01	.01	0	0	0
4-16	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.03	0	0	0	0	0	0
4-19	*11.0	*13.2	*14.4	.....	12.9	.....	.....	.....	.....	.....	0	0	0	0	0	0	0
4-23	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.47	0	.02	.01	0	0	0
4-28	* 9.2	*10.8	*12.3	.....	10.8	.....	.....	.....	.....	.....	0	0	0	0	0	0	0
5- 3	* 6.0	* 9.1	*12.1	.....	9.1	.....	.....	.....	.....	.....	.18	0	0	0	0	0	0
5- 5	* 6.3	* 8.7	*13.3	.....	9.4	.....	.....	.....	.....	.....	.09	0	0	0	0	0	0
5- 7	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.46	T	0	T	T	T	T
5-11	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.04	0	0	0	0	0	0
5-15	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.09	0	0	0	0	T	T
5-19	4.7	5.7	.....	.....	5.2	10.7	11.5	.....	.....	11.1	.24	0	0	0	0	0	0
5-20	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1.15	0	0	0	0	0	0
5-21	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.44	T	0	T	0	.08	.04
5-22	14.2	11.0	.....	.....	12.6	*14.6	*13.5	.....	.....	14.0	.43	0	.08	.04	.11	.19	.15
5-24	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.21	0	0	0	.01	0	01

*See descriptive notes.

Table 3.—*Continued*

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
5–26	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.63	0	0	0	0	0	0
5–28	13.3	13.2	.....	.....	13.2	*12.6	*13.2	.....	.....	12.9	.40	0	0	0	0	0	0
5–29	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.16	0	0	0	0	0	0
5–30	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1.22	0	.02	.01	.18	* .19	.18
6– 1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.87	T	0	T	.01	.20	.10
6– 2	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1.14	0	.04	.02	.18	.32	.25
6– 4	*12.5	*11.8	12.5	12.3	12.3	11.9	13.6	14.3	14.6	13.6	.01	0	0	0	0	0	0
6– 7	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	T	0	0	0	0	0	0
6–11	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.51	0	.02	.01	0	0	0
6–14	7.7	8.4	.....	.....	8.0	10.7	10.8	.....	.....	10.8	.20	0	0	0	0	0	0
6–15	14.5	12.1	.....	.....	13.3	14.2	14.7	.....	.....	14.4	1.10	0	.02	.01	.13	.13	.13
6–19	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.14	0	0	0	0	.01	.01
6–22	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.06	0	0	0	0	0	0
6–27	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.07	0	0	0	0	0	0
6–30	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.36	0	0	0	0	0	0
7– 2	12.7	6.3	.....	.....	9.5	10.6	8.2	.....	.....	9.4	.86	0	0	0	.04	.26	.15
7– 3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.06	0	0	0	0	0	0
7– 6	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.58	0	0	0	0	.10	.05
7– 8	11.2	12.6	.....	.....	11.9	14.1	13.9	.....	.....	14.0	2.11	.06	.41	.24	.60	.79	.70
7–16	9.7	8.6	.....	.....	9.2	10.4	11.4	.....	.....	10.9	.29	0	0	0	0	0	0
7–19	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.80	.04	.10	.07	.02	.17	.10
7–21	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.44	0	.01	.01	.05	.12	.08
7–29	6.1	8.0	.....	.....	7.0	7.7	10.3	.....	.....	9.0	.66	.07	.08	.08	.10	* .18	.14
8– 7	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.46	.01	0	.01	0	.01	.01
8–18	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	4.22	.61	1.33	.97	1.61	1.73	1.67
8–21	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1.46	.03	0	.02	02	.12	.07
9– 7	14.4	12.0	.....	.....	13.2	13.9	14.7	.....	.....	14.3	2.22	0	0	0	* .24	* .39	.32
9–10	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.75	0	.02	.01	.01	.01	.01
9–25	10.4	10.8	.....	.....	10.6	12.2	12.9	.....	.....	12.6	1.04	.04	.04	04	.05	.07	.06
9–27	15.8	13.3	.....	.....	14.6	17.1	16.8	.....	.....	17.0	1.82	.01	.01	.01	.01	.18	.10
10– 5	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1.30	0	0	0	0	05	02
10–15	12.7	11.3	.....	.....	12.0	15.0	14.2	.....	.....	14.6	0.37	0	0	0	0	0	0
10–18	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.78	0	0	0	0	.10	.05

*See descriptive notes.

Table 3.—*Continued*

Date	Moisture in Soil, in per cent of dry weight										Rainfall in Inches	Runoff, in inches depth					
	Under Sod Cover, depth of					Bare Soil, depth of						Sod			Bare Soil		
	6″	12″	18″	24″	Average	6″	12″	18″	24″	Average		Level	Sloping	Average	Level	Sloping	Average
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
1915																	
10–23	13.0	12.4	.....	.....	12.7	15.8	15.4	.....	.....	15.6	.03	0	0	0	0	0	0
11– 1	9.6	10.5	10.3	12.5	10.7	12.4	14.5	18.5	13.3	14.7	0	0	0	0	0	0	0
11–12	8.8	9.5	12.1	.....	10.1	13.7	17.5	16.8	.....	16.0	.58	0	0	0	0	0	0
11–19	16.6	14.9	.....	.....	15.8	18.3	19.7	.....	.....	19.0	1.64	0	01	01	0	.01	01
*11–30	15.9	16.2	19.3	.....	17.1	15.9	16.8	19.0	.....	17.2	.18	0	0	0	0	0	0
12–11	15.6	15.5	15.6	.....	15.6	15.5	15.7	18.9	.....	16.7	* .15	0	0	0	0	0	0
12–18	17.1	16.1	18.8	.....	17.3	15.6	15.8	17.6	.....	16.3	1.75	0	.03	.02	.02	.03	.03
1916																	
1– 6	18.0	15.1	16.1	.....	16.4	16.8	18.7	19.7	.....	*18.4	2.50	0	0	0	0	.01	.01
1–14	19.3	20.3	16.6	.....	18.7	14.9	14.1	14.2	.....	*14.4	2.52	.03	.15	.09	.11	.34	.22
1–25	16.0	16.6	.....	18.0	16.9	18.0	14.5	.....	14.7	15.7	.92	* .38	* .36	.37	.01	.04	.02
2– 2	16.4	15.6	.....	17.7	16.6	16.5	15.3	.....	16.8	*16.2	2.39	0	.01	.01	.01	.09	.05
2–17	17.9	16.2	20.4	.....	*18.2	14.4	15.3	14.9	.....	*14.9	1.18	0	0	0	0	0	0
2–28	17.3	17.9	16.2	16.5	*17.0	12.7	14.9	16.8	19.1	*15.9	.10	0	0	0	0	.01	.01
* 3– 9	15.7	16.2	19.3	20.0	17.8	15.2	16.6	18.1	13.7	15.9	.68	.01	0	.01	0	0	0
3–16	19.7	19.7	15.9	18.2	*18.4	19.0	16.0	14.9	18.0	*17.0	* .19	0	.01	.01	0	.02	.01
3–23	19.2	19.6	19.3	21.6	19.9	20.0	16.5	14.3	16.2	16.7	.20	0	.01	.01	0	0	0
4– 1	17.5	15.5	17.7	19.8	17.6	14.6	13.3	15.5	16.0	14.8	2.06	.02	0	.01	.18	.37	.28
4–10	19.1	16.0	17.5	18.5	17.8	15.3	13.6	15.2	14.9	14.8	.65	0	0	0	0	0	0
4–17	17.8	17.9	16.4	18.7	17.7	13.4	12.3	13.4	15.6	13.7	.28	.01	.01	.01	.01	.01	.01
4–22	19.7	19.7	19.3	20.3	19.8	16.5	13.8	14.3	16.5	15.3	1.08	0	.01	.01	.23	.37	.30
4–29	19.1	18.4	17.8	12.0	16.8	14.2	12.0	13.8	13.6	13.4	.76	.02	0	.01	.07	.08	.08
5– 8	15.4	16.3	16.9	17.5	16.5	13.0	11.7	12.1	12.6	12.3	1.81	0	0	0	.60	.58	.59
5–12	13.6	16.1	15.8	16.5	15.5	11.7	10.8	13.8	12.6	12.2	.10	.01	0	.01	*0	*0	0
5–20	6.4	7.4	10.5	11.7	9.0	9.2	9.3	11.1	12.0	10.4	.10	0	0	0	0	0	0

*See descriptive notes.

Table 3.—*Continued*

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
1916																	
5-23	10.7	7.1	9.5	11.6	9.7	12.1	9.5	10.0	11.0	10.6	1.05	0	0	0	.01	0	.01
5-31	11.1	6.1	9.5	10.8	9.4	14.4	14.4	14.7	14.9	14.6	1.07	.02	0	.01	.01	.03	.02
6- 3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	1.99	0	.04	.02	1.19	*1.11	1.15
6-12	12.2	10.8	11.7	12.3	11.8	16.3	16.5	15.6	12.4	15.2	1.89	0	0	0	.09	.20	.14
6-19	16.5	12.7	14.2	14.6	14.5	14.8	13.9	15.1	14.5	14.6	.96	* .04	*0	.02	.04	.11	.08
6-21	14.8	13 9	15.0	15.5	14.8	16.0	13.6	14.6	13.8	14.5	.82	0	0	0	.05	.22	.14
7- 5	4.4	7.0	01.5	11.3	8.3	6.8	7.9	10.0	11.4	9.0	.15	0	0	0	*0	*0	0
7-24	3.7	4.9	10.1	13.6	8.1	2.9	2.9	10.8	3.1	4.9	.32	.01	0	.01	0	0	0
8- 2	3.5	4.7	4.5	6.1	4.7	2.1	2.8	3.5	3.9	3.1	.50	.08	.02	.05	.08	.16	.12
8- 7	14.9	12.8	13.5	12.4	13.4	11.9	5.6	5.0	4.9	6.9	3.63	*0	* .24	.12	1.55	* .97	1.26
8- 8	16.5	15.2	14.3	13.4	14.9	18.0	16.6	14.1	8.3	14.2	.93	.02	0	.01	0	.04	.02
8-21	9.0	9.2	11.0	11.8	10.2	5.2	5.0	4.0	4.7	4.7	.12	0	0	0	0	0	0
8-28	8.0	8.2	10.3	10.6	9.3	3.1	3.3	4.1	4.4	3.7	.80	0	.01	.01	0	.02	.01
9- 6	13.9	15.9	15.3	16.5	15.4	15.1	5.4	6.8	8.7	9.0	4.32	0	.05	.02	*1.90	1.66	1.78
9- 9	16.8	17.3	14.7	15.8	16.2	12.6	10.7	6.5	*	9.9	.49	*0	*0	0	.05	.15	.10
9-22	6.2	7.4	9.8	11.4	8.7	9.1	8.4	6.5	5.4	7.4	.06	*0	*0	0	*0	*0	0
9-29	16.2	17.1	19.7	12.6	16.4	14.3	5.1	13.2	12.6	11.3	1.68	0	0	0	0	0	0
10- 6	10.5	7.5	9.5	9.8	9.3	8.8	8.1	8.2	6.4	7.9	.01	0	0	0	0	0	0
10-13	12.2	11.9	10.2	11.5	11.4	9.8	8.9	9.5	8.5	9.2	.41	.02	0	.01	.01	0	.01
10-23	15.6	17.0	13.8	13.9	15.1	13.8	10.1	9.3	8.5	10.4	1.31	0	0	0	0	0	0
11-10	14.6	14.7	14.7	16.1	15.0	14.4	10.1	9.9	8.7	10.8	.42	0	0	0	0	0	0
11-20	11.0	10.3	11.9	11.9	11.3	13.6	12.4	11.2	9.5	11.7	.23	*0	*0	0	0	0	0
11-24	16.6	16.4	16.3	18.1	16.9	17.7	15.7	14.2	12.8	15.1	.84	0	.01	.01	.01	.01	.01
12- 4	.....	.....	.....	.....	*	15.5	13.9	13.8	10.5	13.4	.55	0	0	0	.09	.01	.05
12-27	19.1	17.4	15.2	17.8	17.4	16.8	15.2	15.4	14.0	15.4	1.94	* .72	* .40	.56	* .40	* .12	.26
1917																	
1- 4	20.4	18.7	17.0	17.3	18.4	19.3	15.4	15.6	17.0	16.8	.27	.02	0	.01	0	0	0
1- 8	17.1	16.0	18.5	20.0	17.9	16.3	13.8	14.5	15.9	*15.1	2.04	*0	*0	0	1.04	.41	.72
1-22	18.3	16.8	15.7	17 2	17.0	19.9	18.0	16.6	15.9	*17.6	1.05	* .04	* .01	.02	* .10	* .05	.08
1-31	19.7	20.0	19.4	20.4	19.9	17.5	15.2	15.7	16.7	*16.3	.32	.01	0	.01	.05	0	.02
2-27	19.6	18.2	17.0	15.6	17.6	21.8	17.2	14.3	14.0	16.8	1.08	.01	.04	.02	.22	.06	.14
3-12	18.6	19.6	19.9	20.9	19.8	17.9	18.9	18.3	16.3	17.8	.70	0	0	0	.01	.02	.02

*See descriptive notes.

Table 3.—*Continued*

Date	Moisture in Soil, in per cent of dry weight										Rainfall in Inches	Runoff, in inches depth					
	Under Sod Cover, depth of					Bare Soil, depth of						Sod			Bare Soil		
	6′	12′	18′	24′	Average	6′	12′	18′	24′	Average		Level	Sloping	Average	Level	Sloping	Average
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
1917																	
3-14	16.2	18.9	20.4	20.4	19.0	19.3	20.6	19.9	21.3	20.3	1.44	.01	.18	.10	.79	.38	.58
3-24	16.7	19.2	19.5	19.7	18.8	19.7	21.6	21.6	21.5	21.1	.82	.01	0	.01	.16	.08	.12
4- 3	19.4	23.3	15.9	17.6	19.0	18 2	16.0	16.4	20.7	17.8	.96	0	0	0	0	0	0
4-14	13.9	12.6	13.8	14.3	13.6	12.0	13.0	14.5	5.0	11.1	.48	.01	0	0	0	0	0
4-20	13.4	12.1	14.2	14.4	13.5	14.3	14.2	14.7	11.6	13.7	.40	0	0	0	.01	.02	.02
4-28	15.6	17.0	18.2	17.4	17.0	20.2	18.0	15.7	16.5	17.6	1.04	0	.01	.01	0	.02	.01
5- 2	17.5	18.5	18.9	19.0	18.5	16.8	14.7	15.6	15.3	15.6	1.00	0	0	0	.46	.47	.46
5- 5	16.5	18.1	20.9	*29.3	18.5	20.2	20.4	21.8	*40.7	20.8	1.23	0	0	0	.09	.11	.10
5-12	14.4	11.7	13.8	13.1	13.2	11.5	12.9	14.0	14.3	13.2	.15	.01	.01	.01	.05	0	.02
5-25	20.8	11.1	12.5	14.3	14.7	12.1	11.0	10.7	11.7	11.4	.33	*0	*0	0	*0	* .01	.01
5-29	15.3	17.1	18.2	16.7	16.8	17.0	14.3	14.7	15.0	15.2	2.28	0	.01	.01	.97	.83	.90
6-11	13.9	13.2	16.8	12.0	14.0	11.4	12.1	12.5	10.9	11.7	2.77	.01	0	.01	1.08	.73	.90
6-22	8.5	8.6	10.2	12.4	9.9	7.6	10.6	11.5	11.7	10.4	0.44	0	0	0	.04	.08	.06
6-30	16.4	18.0	17.7	16.8	17.2	13.5	15.1	12.6	12.1	13.3	2.90	.02	.03	.02	*1.73	1.66	1.70
7- 9	11.9	11.4	11.0	8.1	10.6	11.9	9.9	9.8	9.9	10.4	1.36	.01	.01	.01	.25	.34	.30
7-14	14.3	13.9	14.4	15.4	14.5	13.3	12.7	13.6	12.7	13.1	.84	.02	0	.01	.21	* .21	.21
7-18	14.8	12.9	14.0	14.1	14.0	13.4	12.5	12.9	11.1	12.5	1.43	0	.01	.01	.01	.79	.40
7-28	16.3	17.6	18.5	15.4	17.0	12.2	12.6	13.2	13.9	13.0	.96	0	.02	.01	.25	.60	.42
8- 7	8.9	12.2	13.2	13.1	11.8	10.8	9.8	10.2	10.3	10.3	.37	0	.01	.01	0	.11	06
8-13	6.8	9.4	10 9	10.5	9.4	8.5	10.6	12.1	14.4	11.4	.19	0	0	0	0	0	0
8-25	15.3	16.7	10.7	9.6	13.1	10.0	6.1	6.7	7.1	7.5	3.04	.03	.01	.02	1.35	1.52	1.44
8-31	9.7	8.2	8.5	8.5	8.7	12.3	13.5	11.6	7.5	11.2	.46	0	0	0	.01	.02	.02
9- 8	13.0	12.1	7.5	9.3	10.5	13.4	14.2	14.7	14.0	14.1	1.15	0	0	0	.09	.10	.10
9-17	12.5	12.8	18.4	15.0	14.7	10.3	12.0	11.7	11.7	11.4	0	0	0	0	0	0	0
9-22	12.9	14.4	13 1	11 7	13 0	12 6	12 9	11 4	11.0	12.0	.16	0	0	0	0	0	0

*See descriptive notes.

Table 3.—*Continued*

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
1917																	
9-28	8.3	11.1	11.9	14.4	11.4	9.5	9.5	11.1	10.5	10.1	.22	0	0	0	0	.01	.01
10- 6	6.0	8.0	10.2	12.5	9.2	9.5	11.8	12.0	11.1	11.1	.19	0	0	0	0	0	0
10-16	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.46	.02	0	.01	0	0	0
10-19	18.3	23.2	10.9	12.5	16.2	17.7	18.1	15.7	12.6	16.0	1.56	0	.01	.01	.49	.59	.54
10-23	17 8	11.6	9.6	10.2	12.3	17.1	17.8	16.0	13.3	16.0	.28	0	0	0	0	0	0
10-30	18.3	17.4	14.0	15.9	16.4	16.8	19.7	19.0	17.2	18.2	1.40	0	0	0	.59	.51	.55
11- 5	16.8	16.3	13.5	13.8	15.1	18.4	19.5	16.7	14.2	17.2	02	0	0	0	0	0	0
11-13	17.6	23.0	12.9	12.9	16.6	16.0	16.4	17.0	15.0	16.1	T	0	0	0	0	0	0
11-26	15.1	14.0	13.4	15.9	14.6	10.8	15.1	15.8	14.3	14.0	.03	0	0	0	0	0	0
* 12- 3	17.0	13.9	12.4	13.8	14.3	14.2	17.2	16.4	14.5	15.6	.30	0	0	0	0	.01	.01
1918																	
* 2-11	24.8	20.9	16.5	15.7	19.5	21.8	20.2	18.5	15.5	19.0	*5.26	*2.29	*2.74	2.52	*3.05	*2.56	2.80
2-26	18.7	21.0	18.4	15.9	18.5	18.6	21.8	21.0	14.3	18.9	2.54	0	.16	.08	1.47	.37*	.92
3- 8	18.6	16.0	14.4	17.5	16.6	18.5	19.0	18.7	16.1	18.1	.65	0	0	0	0	0	0
4-12	18.5	14.6	12.9	13.8	15.0	17.6	16.4	18.3	20.0	18.1	2.57	0	0	0	.26	.15	.20
4-24	17.8	18.0	17.0	17.3	17.5	18.3	18.0	16.6	15.6	17.1	1.50	.01	0	.01	.32	.27	.30
5- 1	17.5	13.7	14.5	15.5	15.2	16.2	17.9	16.7	16.1	16.7	1.05	0	.01	.01	.31	.21	.26
5- 8	24.2	15.9	16.6	14.8	17.7	14.0	20.1	14.5	14.5	15.8	.19	0	.01	.01	0	.01	.01
5-14	19.3	19.1	19.6	18.9	19.2	19.5	18.5	18.6	17.1	18.4	2.77	.02	.01	.02	.95	* .75	.85
5-21	13.9	15.2	17.6	16.7	15.8	12.5	11.3	12.3	13.3	12.4	.38	0	0	0	0	.01	.01
5-29	10.9	11.4	11.3	11.7	11.3	11.7	11.1	10.6	11.5	11.2	.29	0	0	0	0	.04	.02
6- 1	13.8	13.9	13.3	13.0	13.5	11.0	10.6	10.4	11.4	10.8	.23	.01	.01	.01	.01	.04	.02
6- 6	15.0	12.9	13.9	12.5	13.6	15.4	11.4	10.9	12.3	12.5	1.43	0	0	0	.23	.33	.28
6-15	14.0	13.0	11.8	11.4	12.6	11.6	10.9	10.9	10.8	11.0	1.05	0	.01	.01	.47	.52	.50
* 6-26	11.1	7.7	8.5	11.2	9.6	12.2	8.9	9.7	10.8	10.4	.94	.02	0	.01	.21	.21	.21
7- 1	15.3	18.3	15.9	15.6	16.3	14.1	12.6	8.3	8.3	10.8	1.30	0	.01	.01	.18	.57	.38
7-11	9.1	10.6	8.8	12.1	10.2	5.9	7.4	8.7	8.8	7.7	0	0	0	0	0	0	0
7-12	10.6	13.8	13.6	12.9	12.7	5.3	6.4	7.4	8.5	6.9	.22	0	0	0	0	0	0
7-17	8 8	9.4	11.8	11.3	10.3	6.8	4.4	6.1	*	5.8	.37	0	0	0	0	01	.01
7-23	16.0	17.2	16.2	16.1	16.4	13.0	6.1	6.7	10.3	9.0	3.11	.03	.11	.07	2.23	2.20	2.22
7-25	15.1	16.9	16.7	15.8	16.1	16.2	12.7	10.7	13.2	13.2	.57	0	0	0	.18	* .04	.11
8- 9	6.1	5.5	5.5	6.1	5.8	4.5	6.8	6.9	7.9	6.5	.27	0	0	0	.01	0	.01

*See descriptive notes.

Table 3.—*Continued*

Date	Moisture in Soil, in per cent of dry weight										Rainfall in Inches	Runoff, in inches depth					
	Under Sod Cover, depth of					Bare Soil, depth of						Sod			Bare Soil		
	6″	12″	18″	24″	Average	6″	12″	18″	24″	Average		Level	Sloping	Average	Level	Sloping	Average
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
1918																	
8-13	12.2	15.5	16.0	14.5	14.6	7.9	6.3	5.4	5.6	6.3	1.24	.02	.02	.02	.61	.71	.66
8-23	*	14.5	16.3	14.3	15.0	15.2	16.3	16.5	15.3	15.8	1.06	0	0	0	.04	.33	.18
8-28	14.0	12.3	9.0	8.2	10.9	*	*	5.6	6.3	6.0	1.54	.01	.01	.01	.01	.12	.06
9- 3	15.3	19.5	19.6	17.8	18.0	12.3	13.3	14.4	10.9	12.7	1.39	0	0	0	.37	.78	.58
9- 6	15.9	17.8	19.6	19.3	18.1	13.5	15.8	13.8	10.9	13.5	.62	0	0	0	.07	.12	.10
9-26	15.4	16.1	18.6	17.9	17.0	13.9	16.9	16.1	13.9	15.2	3.54	0	.01	.01	1.17	1.24	1.20
10- 3	13.6	13.4	17.3	18.3	15.6	14.3	14.8	15.6	15.8	15.1	.01	0	0	0	0	0	0
10- 9	10.2	12.4	17.5	17.6	14.4	13.7	15.7	14.9	13.7	14.5	.10	0	0	0	0	0	0
10-21	12.5	15.5	18.4	17.4	16.0	14.5	15.2	14.6	13.0	14.3	.49	0	0	0	0	0	0
10-28	16.7	16.5	17.2	16.5	16.7	17.0	17.3	15.4	14.9	16.1	.99	0	.01	.01	.13	.30	.22
11- 1	15.8	15.7	17 8	17.2	16.6	16.4	17.9	16.9	15.4	16.7	.18	0	0	0	0	0	0
11- 7	14.5	15.3	17.0	16.7	15.9	16.1	17.0	16.7	15.3	16.3	0	0	0	0	0	0	0
11-15	14.4	14.7	16.6	16.2	15.5	12.7	15.4	15.3	14.2	14.4	.20	0	0	0	0	0	0
11-21	17.5	16.4	17.3	16.5	16.9	16.0	18.4	17.0	14.8	16.6	.89	0	0	0	0	0	0
11-30	17.6	17.7	19.6	17.8	18.2	17.3	18.3	16.9	15.3	17.0	.73	0	0	0	0	.13	.16
12- 7	15.7	17.8	18.1	18.2	17.4	15.8	18.1	16.6	17.0	16.9	.12	0	0	0	0	0	0
12-13	18.8	20.4	20.4	20.2	20.0	19.6	21.4	18.8	17.3	19.3	1.59	0	.01	.01	.07	.23	.15
12-19	17.7	17.7	19.9	18.1	18.4	18.1	19.2	17.3	15.5	17.5	.44	0	.01	.01	.01	0	.01
12-31	19.4	19.2	19.3	18.3	19.0	19.9	18.4	16.3	15.6	17.6	.62	0	0	0	0	0	0
1919																	
1-16	20.0	18.5	18.3	17.7	18.6	21.1	19.6	18.5	18.0	19.3	.85	0	0	0	0	0	0
1-24	17.6	18.3	23.2	20.3	19.8	21.3	19.7	17.5	16.6	18.8	.47	0	0	0	0	0	0
2- 4	17.3	18.1	22.0	20.0	19.3	18.1	18.6	16.8	15.7	17.3	.06	0	0	0	0	0	0
2-11	15.7	19.1	21.4	20.5	19.2	17.9	16.6	16.9	15.5	16.7	.02	0	0	0	0	0	0
2-17	17.6	18.8	22.4	20.3	19.8	20.8	19 4	17.4	16.2	18.4	.68	0	0	0	.01	.01	.01

*See descriptive notes.

Table 3.—*Continued*

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
1919																	
2–25	17.5	20.1	22.5	21.0	20.3	20.0	20.2	18.5	16.4	18.8	.26	0	0	0	0	0	0
3– 3	16.8	17.8	22.8	22.1	19.9	18.0	18.6	18.8	16.4	18.0	.09	0	0	0	0	0	0
3–10	18.2	19.7	22.8	23.4	21.0	19.0	21.5	20.0	17.8	19.6	1.09	0	0	0	0	0	0
3–13	18.4	18.4	22.1	22.5	20.4	17.6	19.4	16.7	15.1	17.2	.04	0	0	0	0	0	0
3–18	19.6	20.3	21.4	21.1	20.6	18.8	20.5	18.3	16.2	18.4	2.97	0	.03	.02	.04	.07	.06
3–26	17.1	18.1	19.9	19.3	18.6	16.1	19.0	18.2	17.5	17.7	.10	0	0	0	0	0	0
3–27	18.1	21.6	22.6	20.3	20.6	19.2	20.3	20.1	17.4	19.2	.85	0	0	0	0	0	0
4– 5	20.0	18.6	18.1	17.9	18.6	18.5	19.3	17.4	16.7	18.0	.09	0	0	0	0	0	0
4–11	21.5	21.1	18.5	17.3	19.4	19.4	20.4	17.5	16.0	18.3	1.53	0	.03	.02	.13	.16	.14
4–15	16.9	20.6	16.3	17.0	17.7	21.7	20.1	18.7	17.9	19.6	.30	0	.01	.01	0	.01	.01
4–18	21.1	20.5	18.4	17.3	19.3	16.0	20.0	18.0	17.1	17.8	1.02	.01	.01	.01	.13	.14	.14
4–25	18.8	17.9	15.0	15.3	16.8	17.5	17.2	15.3	14.1	16.0	.12	0	0	0	0	0	0
5– 1	21.0	20.7	18.6	17.0	19.3	19.5	20.2	21.9	20.6	20.6	1.06	0	.01	.01	.08	.08	.08
5– 7	21.3	21.1	19.2	17.0	19.6	19.6	20.5	21.6	20.5	20.6	.73	0	0	0	.17	.26	22
5–12	20.5	19.7	17.7	17.3	18.8	20.7	21.6	21.1	19.7	20.8	1.75	0	.02	.01	.10	.12	.11
5–21	20.6	20.1	17.6	16.9	18.8	20.0	20.8	20.3	18.3	19.8	.84	0	0	0	.12	.02	.07
5–29	17.1	17.2	15.6	15.2	16.3	16.3	17.9	19.3	16.8	17.6	.80	0	0	0	.04	.05	.04
6–12	15.0	15.2	14.7	14.8	14.9	17.2	19.5	18.9	17.7	18.3	.70	0	0	0	.09	.14	.12
6–18	19.9	19.1	16 9	16.3	18.0	17.7	21.1	19.5	19.8	19.5	1.73	.01	.06	.04	.84	.97	.90
7– 1	12.2	14.0	13.8	14.1	13.5	17.1	18.9	19.6	17.5	18.3	.38	0	0	0	.01	0	.01
7– 9	7.1	10.3	11.9	13.2	10.6	14.5	16.2	17.6	16.4	16.2	0	0	0	0	0	0	0
7–16	12.2	11.7	12.2	13.4	12.4	16.2	19.8	18.3	17.1	17.8	1.51	0	0	0	.20	.49	.34
7–21	16.9	16.9	14.7	13.8	15.6	18.1	21.4	20.0	18.8	19.6	1.66	0	.01	.01	.18	.01	.10
7–28	11.9	14.7	14.2	14.2	13.8	17.3	20.5	18.5	17.2	18.4	0	0	0	0	0	0	0
7–31	9.0	10.2	10.7	11.5	10.4	23.2	17.7	17.9	17.5	19.1	.81	0	.01	.01	.04	.09	.06
8– 4	9.8	10.6	12.1	12.7	11.3	16.0	17.3	18.1	16.9	17.1	.05	0	0	0	.01	0	.01
8–21	17.9	10.2	8.3	9.5	11.5	20.7	18.4	19.3	17.1	18.9	2.22	.01	.04	.02	.56	.88	.72
8–25	20.6	19.0	18.4	16.9	18.7	16.6	20.5	18.6	15.9	17.9	2.56	0	.34	.17	1.70	1.74	1.72
9– 2	18.9	18.3	17.1	16.6	17.7	14.0	19.2	18.9	16.3	17.1	.74	0	.06	.03	.30	.27	.28
9–15	13.0	13.3	12.7	12.3	12.8	12.7	14.1	14.2	13.8	13.7	.46	0	0	0	.04	.08	.06
10– 3	10.1	11.6	11.1	12.3	11.3	10.9	14.9	13.8	13.3	13.2	.88	* 0	*.01	.01	0	0	0
10– 7	12.4	11.3	10.5	11.0	11.3	12.5	14.4	13.6	12.9	13.4	.60	0	.01	.01	0	.01	.01
10–15	16.2	15.4	14.3	13.8	14.9	19.3	20.6	16.9	15.7	18.1	2.33	0	.03	.02	.14	.11	.12
10–29	15.5	14.5	13.5	14.7	14.6	19.7	20.9	18.1	15.8	18.6	2.74	0	.04	.02	.54	.41	.48

*See descriptive notes.

moisture under the bare surface are connected by dotted lines. These connecting lines have been drawn to aid the reader in studying the soil moisture under a given surface cover, from point to point across the diagrams. No attempt has been made to show the daily fluctuations. The lines have been drawn practically straight from point to point, whereas, in the case of an increase in the amount of moisture, the line, in order to show the conditions accurately, should have been drawn on a slightly downward slope until the day of heavy rainfall and should then have risen more or less abruptly to the higher percentage. Although the diagrams were prepared by platting percentages as ordinates against dates as abscissas, scales showing the actual amounts of water in the soil in inches depth, for the two foot depth of soil involved, have been added at the edges of the sheets.

Notes to Accompany Table 3

On account of the condensed form of table 3 it has not been possible to include descriptive notes. Since such notes are important in any study of the individual records, they are reproduced, herewith, arranged chronologically so that any date can be easily located. Asterisks have been inserted in table 3 after the quantities for which descriptive notes are available.

March 6, 1915.—Average percentage based on two samples taken at this depth.

March 16, 1915.—Average percentage based on three samples taken at this depth. Values given for depths of 18 and 24 inches are questionable and have not been used in computing the average.

April 19 and 28, 1915.—Average percentage based on two samples taken at this depth.

May 3, 5, 22, and 28, 1915.—Average percentage based on two samples taken at this depth.

May 30, 1915.—Runoff was some greater than value given, due to clogging of sewer pipe.

June 4, 1915.—Average percentage based on two samples taken at this depth.

July 29, 1915.—Runoff was some greater than value given, due to clogging of sewer pipe.

Sept. 7, 1915.—Ground within these plats spaded thoroughly to a depth of about six inches.

Nov. 30, 1915.—Light snow on ground. Ground frozen to a depth of about one inch.

Dec. 11, 1915.—Sleeting at time of observation.

Jan. 6, 1916.—Soil under bare surface frozen to a depth of about three inches.

Jan. 14, 1916.—Soil under bare surface frozen to a depth of about five inches.

Jan. 25, 1916.—Runoff due to rain falling on frozen saturated soil surface.

Feb. 2, 1916.—Soil under the bare surface frozen to a depth of about two inches.

Feb. 17, 1916.—Soil under the bare surface frozen to a depth of from one to four inches, but soft on top. Soil under sod not frozen.

Feb. 28, 1916.—Soil under the bare surface frozen to a depth of about five inches. Soil under sod frozen to a depth of about four inches.

Mar. 9, 1916.—Ground thawing out.

Mar. 16, 1916.—Soil under bare surface frozen to a depth of about a half an inch. Soil under the sod frozen to a depth of about an inch. About six inches of snow on the ground.

May 12, 1916.—Ground within these plats spaded thoroughly to a depth of about four inches.

June 3, 1916.—Record probably low due to runoff overtopping side of plat at lower corner.

June 19, 1916.—Surfaces within these plats covered with a dense growth of white clover and bluegrass about twelve inches high.

July 5, 1916.—Grass and weeds removed from these plats, and ground spaded.

Aug. 7, 1916.—Runoff from sloping plat with bare soil surface probably low due to leakage. Grass in sod covered plats about six inches high.

Sept. 6, 1916.—Runoff record for level bare soil plat probably too low due to can overflowing.

Sept. 9, 1916.—Gravel encountered at depth of twenty-four inches under bare surface. Blue grass in sod covered plats three to six inches high.

Sept. 22, 1916.—Ground within bare soil plats spaded to a depth of about three inches. Blue grass in sod covered plats about five inches high.

Nov. 20, 1916.—Grass in sod covered plats about two inches long and dying.

Dec. 4, 1916.—Samples not taken under sod on account of rain.

Dec. 27, 1916.—Runoff due to melting snow, and to rain falling on frozen ground.

Jan. 8, 1917.—Soil not frozen. Dead grass from one to five inches long in sod covered plats.

Jan. 22, 1917.—Soil under bare surface frozen to a depth of about three inches. Bare soil plats about half covered with snow about a half an inch deep. No snow on sod plats.

Jan. 31, 1917.—Soil not frozen.

May 5, 1917.—Data questionable, results not used in computing averages.

May 25, 1917.—Grass two to twelve inches long on sod covered plats. Weeds removed from bare soil plats.

June 30, 1917.—Record may be slightly low due to leakage.

July 14, 1917.—Record uncertain due to clogging of tile entrance.

Dec. 3, 1917.—Owing to the unusually severe winter weather no records were taken during the remainder of this month or during the month of January, 1918. The total precipitation in January, two-thirds of which was snowfall, was 3.46 inches at the Dayton U. S. Weather Bureau Station and 3.87 inches at the Dayton cooperative station.

Feb. 11, 1918.—Snow and ice practically gone, ground frozen to a depth of about eight inches. Records of runoff uncertain due to overtopping of cans and due to water entering plats from outside snow accumulations. Precipitation measurements uncertain; value of 5.26 inches given was observed at the Dayton cooperative station.

Feb. 26, 1918.—Record low due to can overflowing.

May 14, 1918.—Record probably low due to clogging of tile entrance.

June 26, 1918.—Placed wire mesh over entrance to tiles.

July 17, 1918.—Sample disturbed in drying.

July 25, 1918.—Record probably low due to leakage around tile.

Aug. 23 and 28, 1918.—Sample disturbed in drying.

Oct. 3, 1919.—Grass about eight inches long on level sod covered plat, and about two to six inches long on sloping sod covered plat.

It is believed that all records of soil moisture obtained during the summer and fall of 1919 are slightly high compared with the preceding records, due either to taking the samples a little farther away from the plats, where the soil and topography were slightly different, or to getting them more thoroughly dried.

Accuracy of the Data

The data is believed to be sufficiently accurate for the purposes for which it was collected. Although difficulties were encountered, particularly during the first year, they were finally overcome in most instances.

The soil samples were taken for the purpose of learning about how much water was in the ground before and after rains, especially when runoff occurred, rather than for making thorough studies of soil moisture. Consequently the records should not be used indiscriminately in any detailed study of the subject. Any individual value given in table 3 may be considerably in error. Due to the difficulties encountered in taking, drying, and weighing the samples, no single percentage is probably accurate to within less than one per cent; that is, a value given as 5 might actually be 4 or 6, or a value given as 20 might actually be 19 or 21. Possibly a few of the minimum records are low due to not getting the samples entirely dry. It must also be remembered that on account of the slight differences in soil texture and the variations in surface configuration within the limited area in which the samples were taken, samples taken on different dates may not be strictly comparable. Probably this effect is even more important than the errors in observation. These conditions, however, are not so important in considering the average moisture content of the upper two feet of soil, as given in columns 6 and 11 of table 3.

The precipitation records are believed to be accurate in all cases except where the greater part of the precipitation occurred as snow. The two rain gages were well located with respect to obstructions and the readings generally checked to within .05 of an inch.

The runoff records for the level plats and for the sloping sod plat are believed to be as accurate as the precipitation records, except where the runoff was caused by the melting of large quantities of snow. There is no doubt but that all of the runoff from these plats entered the runoff tanks and that the amounts were accurately measured. The depths in the runoff tanks could

easily be measured to eighths of an inch, corresponding approximately to hundredths of an inch on the plats. The records for the sloping bare soil plat are somewhat uncertain in several instances, as indicated in the preceding notes.

SOIL MOISTURE

The records of soil moisture, given in table 3, furnish interesting information regarding the conditions at this particular location and pertaining to this particular soil. From the percentages given in table 3 and the weight per cubic foot of the soil, given in table 2, it is possible to discuss the dryest condition which the soil ever reaches, the maximum amount of water that the soil can contain, the maximum amount that it can hold against the force of gravity, the variations throughout the year, the amount of water absorbed during rains, and the rate at which the ground drys out after the rain ceases.

Minimum Records

A study of the records given in table 3 shows that during the length of time covered by the observations the soil was dryest on August 2, 1916. The determinations made on that date showed an average moisture content of only 4.7 per cent for the soil under the sod covering and only 3.1 per cent for the soil under the bare surface, amounts corresponding to 1.80 and 1.19 inches, respectively, for the depth of two feet in which the samples were taken. Although these values may be slightly low due to not getting the samples thoroughly dried, it is known from other information that the ground at this time was baked hard and was very dry, probably as dry as it ever gets. The water in the soil was probably all hygroscopic water. It is unlikely that any further appreciable evaporation or transpiration could take place. Practically all vegetation, including the larger bushes and trees, had been wilting for several days. The wilting coefficient, calculated from the mechanical analyses given previously, by the methods explained on page 69 of Bulletin 230 of the Bureau of Plant Industry,* would be about 12.3 per cent for the sample taken near the level plats and about 13.7 per cent for the sample taken near the sloping plats. Although values as high as these, and higher, are given for loam and clay loam

*The Wilting Coefficient for Different Plants and its Indirect Determination, by Lyman J. Briggs and W. L. Shantz, Bulletin 230 of the Bureau of Plant Industry, U. S. Department of Agriculture, 1912.

soils, in the above mentioned publication, these values seem slightly high for the Moraine Park soil. Observations showing values smaller than these were made at several times when no evidences of wilting could be detected and when there was no reason to believe that the records might be low. According to Bulletin 230 the values of the hygroscopic coefficient would be 0.68 times the values of the wilting coefficient or about 8.4 and 9.3 per cent respectively for the two samples.

The records at the different depths on August 2, 1916, were as follows:

	Depth in Inches				
	6	12	18	24	Ave.
Moisture under sod, percent	3.5	4.7	4.5	6.1	4.7
Moisture under bare surface, percent	2.1	2.8	3.5	3.9	3.1

It will be noticed that the amount of moisture in the soil increased as the depth increased, for both types of surface covering; also that the amount under the sod was greater, at each depth, than the amount under the bare surface. The percentages at the different depths were, themselves, minimum values for the entire period of record, in all cases except at the depth of 24 inches under the bare surface where a value of only 3.1 per cent was obtained on July 24, 1916, the preceding date on which samples were taken. It is probable, however, that the soil at this depth was actually drier on August 2 than on July 24, and that the opposite condition shown by the data is due to errors in observation or in securing comparable samples.

The amount of moisture in the soil was also very low in August, 1918, the measurements of August 19 showing the following percentages:

	Depth in Inches				
	6	12	18	24	Ave.
Moisture under sod, percent	6.1	5.5	5.5	6.1	5.8
Moisture under bare surface, percent	4.5	6.9	6.9	7.9	6.5

It will be noticed that on this date there seemed to be a little more soil moisture under the bare surface than there was under the sod. Under the bare surface the percentage of moisture seemed to increase with the depth, while under the sod it seemed to be about the same at all depths.

It is interesting to note that Widstoe and McLaughlin in their experiments in Utah,* found that in one instance the amount of moisture in the first foot of soil on which crops were

*The Movement of Water in Irrigated Soils, by J. A. Widstoe and W. W. McLaughlin, Bulletin 115 of the Utah Agricultural College Experiment Station, Logan, Utah, May, 1912.

growing was reduced to 5.64 per cent, 40 days after irrigation; and that the amount in the first foot under the bare surface was reduced only to 18.6 per cent, 36 days after irrigation. The value of 5.64 per cent is only about one and a half per cent greater than the minimum Moraine Park record obtained in the first foot of soil under a blue grass sod. However, the value of 18.6 per cent is rather large compared with the value of about 2.5 per cent obtained under the bare surface at Moraine Park. Although there are some differences in soil texture, the real reasons for this wide difference in evaporation are probably the greater percentage of voids in the Utah soil and the differences in the climatic conditions at the two locations. At Moraine Park the percentage of voids in the soil, by volume, is only about 41 while in Utah, where the above experiments were made, it is about 55. In Utah the climate is arid, while in Ohio it is humid. The differences in soil evaporation due to differences in climate were discussed by Buckingham in 1907.* He showed that a moist bare soil in an arid climate dries out rapidly at the surface at first, forming a sort of a dry soil mulch, after which it dries out very slowly; that a moist bare soil in a humid climate dries out less rapidly than in the arid climate at first, so that the dry mulch effect is not produced, and more rapidly later on; the net result being that after several days more water had evaporated from the soil under humid conditions than had evaporated from the soil under arid conditions.

Maximum Records

The maximum percentages of moisture at the different depths, as shown by the data in table 3, occurred on different dates, although some uncertainty exists in this connection due to the difficulties encountered in securing comparable samples. The actual maximum values, not considering a few erratic observations which have been mentioned in the notes as being questionable, are as follows:

	Depth in Inches			
	6	12	18	24
Moisture under sod, percent	24.8	23.3	23.2	23.6
Moisture under bare surface, percent	23.2	21.8	21.9	21.5

These values seem to indicate that the soil under the sod at a given depth never contains more than about 24 per cent of moisture, and that the soil under the bare surface never contains

*Studies on the Movement of Soil Moisture, by Edgar Buckingham, Bulletin 38 of the Bureau of Soils, U. S. Department of Agriculture, 1907.

more than about 22 per cent. In the preceding discussions it was shown, by calculations based on the specific gravity of the soil particles and the determinations of the unit weight of the soil in place, that the soil would be saturated when it contained an amount of moisture equal to about 25.6 per cent of its dry weight, an amount slightly greater than those given above.

The records seem to indicate that the total amount of moisture in the upper two feet never is more than about 21 per cent of the dry weight of the soil, an amount equivalent to a depth over the surface of 8.06 inches. Samples were taken at several times during the months of January, February, and March, when the soil was probably as nearly saturated as it ever becomes under field conditions. The slight differences in moisture content at the same depth shown by the data at such times are probably due to the difficulties encountered in securing comparable samples or in weighing and drying those taken. The observations which gave the maximum average values for the upper two feet are as follows:

	Depth in Inches				
	6	12	18	24	Ave.
Moisture under sod, percent	18.2	19.7	22.8	23.4	21.0
Moisture under bare soil, percent	19.7	21.6	21.6	21.5	21.1

The values for the soil under the sod were obtained on March 10, 1919. Those for the soil under the bare surface were obtained on March 24, 1917. On these dates the percentage of moisture seemed to be slightly greater at the greater depths under the sod, but did not differ materially at the different depths under the bare surface.

The average value of 21 per cent shown by the above data probably represents the maximum amount of water that can be held by the Moraine Park soil; that is, the maximum amount of moisture that can be present without any appreciable downward percolation due to gravity taking place,—the quantity frequently referred to as the "moisture-holding capacity." That this is true is indicated, in a way, by the observations of January 24 and February 4 and 11, 1919. The average percentages of moisture found on these dates were as follows:

	Sod	Bare
January 24, 1919	19.8	18.8
February 4, 1919	19.3	17.3
February 11, 1919	19.2	16.7

The total loss in moisture in the 2-foot depth during the 18 days from January 24 to February 11, indicated by these per-

centages, would be equivalent to a depth in inches of 0.23 for the sod and 0.80 for the bare soil. The total precipitation during this period was 0.08 of an inch, thus increasing the amounts of moisture to be accounted for to 0.31 and 0.88 inches, respectively, or to 0.017 and 0.049 inches per day. As the weather during the greater part of this period was clear with temperatures above freezing and some wind blowing, it is quite likely that these amounts represent soil evaporation alone and that consequently no material percolation occurred.

While the amount of moisture that can be held by the soil undoubtedly varies widely with its composition it is interesting to note that Widstoe and McLaughlin, in their investigations in Utah, previously referred to, found that the maximum amount of water that could be held by the Greenville soil under field conditions was a little less than 24 per cent.

Variations in Soil Moisture

The variations in the amount of moisture in the soil at Moraine Park throughout the year are shown graphically by the curves in the lower parts of figures 4 to 8, inclusive. The amount of moisture under the sod is shown by the continuous lines and the amount under the bare surface is shown by the dotted lines.

A study of these diagrams shows that the soil is generally dryest in the late summer or early fall, during the months of July, August, or September; and wettest in the late winter or early spring, during the months of January, February, or March. It has already been pointed out that the minimum val-

Table 4.—Maximum Percentages of Moisture in the Upper Two Feet of Soil at Moraine Park During the Months of June, July, and August

Year	Moisture under Sod, in %			Moisture under Bare Soil, in %		
	June	July	August	June	July	August
1915	13.3	11.9	.....	14.4	14.0	.....
1916	14.8	8.3	14.9	15.2	9.0	14.2
1917	17.2	17.0	13.1	13.3	13.1	11.4
1918	13.6	16.4	15.0	12.5	13.2	15.8
1919	18.0	15.6	18.7	19.5	19.6	18.9

ues for the entire period of record were obtained in the month of August, and that the maximum values were obtained in the month of March. The curves also show that the amount of

Table 5.—Minimum Percentages of Moisture in the Upper Two Feet of Soil at Moraine Park During the Months of January, February, and March

Year	Moisture under Sod, in %			Moisture under Bare Soil, in %		
	January	February	March	January	February	March
1916	16.4	16.6	17.8	14.4	14.9	15.9
1917	17.0	17.6	18.8	15.1	16.8	17.8
1918	.....	18.5	16.6	.....	18.9	18.1
1919	18.6	19.2	18.6	18.8	16.7	17.2

moisture gradually increases in the fall, during the months of October, November, and December; that it does not change much during the winter months, even in the absence of rainfall; and that it begins to diminish appreciably in the spring, during the months of April or May, due to the requirements of plants and the higher rates of soil evaporation, both of which are brought about by the higher temperatures.

In the summer months the moisture absorbed during rains is rapidly consumed by transpiration and soil evaporation, as soon as the rain ceases, until the ground becomes so dry that capillary movement of the moisture practically ceases or until the amount of available moisture is replenished by additional rainfall. The rates of soil evaporation and transpiration are so high that the upper two feet of soil at Moraine Park seldom, if ever, becomes filled with capillary water during the months of June, July, and August, even though the rainfall may be considerably greater than normal. The maximum percentages found during these months, shown by the data in table 3, are given in table 4.

It will be noticed that the maximum amount of capillary water that the soil can contain, shown by the preceding discussions to be about 21 per cent, was not reached during any one of the months given in table 4; although the percentages were rather high in the case of the bare soil in the summer of 1919. However, it is believed that the records obtained during the summer and fall of 1919 are slightly high compared with those taken previously. The rainfall was considerably greater than normal during the month of July, 1915, when it amounted to 5.80 inches; during the month of August, 1916, when it amounted to 5.98 inches; and during the month of June, 1917, when it amounted to 6.11 inches, the normal amounts for these months at the Dayton Weather Bureau station being 3.28, 3.01, and 3.96 inches respectively.

During the months of January, February, and March the amount of moisture in the soil, even under the most favorable conditions, seldom gets much below the maximum capillary amount, since plant requirements are nil and soil evaporation is very low. The minimum percentages obtained during these months are given in table 5. Records obtained in March, 1915, are not included since the work had hardly become organized at that time.

It will be noticed that while these values are all somewhat lower than the maximum capillary value of 21 per cent, they are all considerably higher than the minimum values of from 3 to 10 per cent which generally occur during the summer months. No records were obtained during the month of January, 1918, due to the unusually severe winter weather at that time. It is known from other observations, however, that the upper foot of soil became practically saturated during the period from December 21 to 29 due to the melting of about 9 inches of snow; also that the ground froze before this water could percolate to a greater depth, and remained frozen until the thawing period which began February 6.

It will be noticed from the curves in figure 8 that during the months of January and February, 1919, there was little change in the amount of moisture in the soil. Very little drying out seemed to take place between rains although the conditions were probably as favorable for the drying out of the soil as they ever are in the winter. The soil was not frozen; the mean temperatures were comparatively high, being about four degrees above normal; and there was some wind blowing the greater part of the time.

The curves in figures 4 to 8, inclusive, show that the changes in the percentages of moisture in the soil between successive observations were considerably greater during the summer months than they were during the winter months, as, of course, would naturally be expected. An inspection of the data in table 3 shows that while the individual observations vary greatly, partly due to differences in soil texture and to errors of observations, the moisture content of the first 6-inch layer of soil seems to vary more than that of the deeper layers. Interesting data on variations in soil moisture at different depths was obtained near Akron, Colorado, by H. L. Shantz, in the summer of 1909.*

*Natural Vegetation as an Indicator of the Capabilities of Land for Crop Production in the Great Plains Area, by H. L. Shantz, Bulletin 201 of the Bureau of Plant Industry, U. S. Department of Agriculture, 1911, page 31.

It was there found that during the period from June 10 to September 10 the rainfall did not affect the moisture content of the soil below a depth of 18 inches, although on July 7 the rainfall amounted to 2.40 inches. However, a rainy period during the last of May and the first part of June had some effect on the moisture content of the soil down to depths of about 3 feet. At Moraine Park, the moisture content of the soil from 18 to 24 inches below the surface seemed to be affected by the rainfall at times during the summer. This difference in depth of penetration at the two locations is probably due to a difference in soil texture.

Records showing variations in soil moisture have been published by numerous investigators. To mention all such data is beyond the scope of this publication. However, the observations by King, published by the Bureau of Soils,* should be referred to since they gave valuable data on the moisture content, at different depths, of eight different soils, under various conditions of cultivation and fertilization, in four different states.

Evaporation and Transpiration

It is interesting to compute the daily rates of evaporation and transpiration for short periods of time from some of the records given in table 3. This has been done for a few selected periods where the data is most reliable. Periods have been chosen in which the rainfall was not excessive, in which there was no appreciable surface runoff, and in which it is believed that there was no percolation of moisture into the underlying beds of sand and gravel. It has been assumed that the decrease in the amount of moisture in the soil in each case, was caused by evaporation and transpiration, and that no moisture was drawn upward by capillary action from the sand and gravel, assumptions which are probably not greatly in error.

The data computed in this manner is given in table 6. The moisture in the soil, in per cent, at the beginning and ending of each period, the amount of water in inches depth corresponding to the decrease in moisture percentage, the total evaporation, and the evaporation in inches per day, are given for the soil under the sod and for the soil under the bare surface. The term evaporation has been used in the table headings to include transpiration as well as soil evaporation. The total rainfall, the num-

*Investigations in Soil Management, by F. H. King, Bulletin 26 of the Bureau of Soils, U. S. Department of Agriculture, 1905, pages 167-191.

Table 6.—Maximum Rates of Evaporation and Transpiration at Moraine Park

Period	Number of Days	Mean Relative Humidity %	Mean Temperature Degrees F.	Rainfall Inches	Under Sod Cover					Under Bare Soil				
					Moisture in Soil		Change in Inches	Total Evaporation Inches	Evaporation per Day Inches	Moisture in Soil		Change in Inches	Total Evaporation Inches	Evaporation per Day Inches
					Begin %	End %				Begin %	End %			
May 8–12, 1916	4	53	63	0.10	16.5	15.5	0.38	0.48	0.12	12.3	12.2	0.04	0.14	0.04
May 12–20, 1916	8	65	58	.10	15.5	9.0	2.50	2.60	.32	12.2	10.4	.69	.79	.10
May 20–23, 1916	3	72	63	1.05	9.0	9.7	—.27	.78	.26	10.4	10.6	—.07	.98	.33
June 21–July 5, 1916	14	66	72	.15	14.8	8.3	2.50	2.65	.19	14.5	9.0	2.11	2.26	.16
July 5–24, 1916	19	69	79	.32	8.3	8.1	.07	.39	.02	9.0	4.9	1.57	1.89	.10
July 24–Aug. 2, 1916	9	65	82	.50	8.1	4.7	1.30	1.80	.20	4.9	3.1	.69	1.19	.13
Aug. 8–21, 1916	13	72	77	.12	14.9	10.2	1.80	1.92	.15	14.2	4.7	3.65	3.77	.29
Aug. 21–28, 1916	7	67	72	.80	10.2	9.3	.34	1.14	.16	4.7	3.7	.38	1.18	.17
Sept. 9–22, 1916	13	66	63	.06	16.2	8.7	2.88	2.94	.23	9.9	7.4	.96	1.02	.08
Sept. 29–Oct. 6, 1916	7	70	59	.01	16.4	9.3	2.73	2.74	.39	11.3	7.9	1.30	1.31	.19
May 5–12, 1917	7	85	45	.15	18.5	13.2	2.03	2.18	.31	20.8	13.2	2.92	3.07	.44
May 14–21, 1918	7	72	69	.38	19.2	15.8	1.30	1.68	.24	18.4	12.4	2.30	2.68	.38
May 21–29, 1918	8	72	74	.29	15.8	11.3	1.73	2.02	.25	12.4	11.2	.46	.75	.09
July 1–11, 1918	10	65	69	.00	16.3	10.2	2.34	2.34	.23	10.8	7.7	1.19	1.19	.12
July 25–Aug. 9, 1918	15	79	79	.27	16.1	5.8	3.96	4.23	.28	13.2	6.5	2.57	2.84	.19
Aug. 23–28, 1918	5	72	74	1.54	15.0	10.9	1.57	3.11	.62	15.8	† 6.0	1.88	3.42	* .68
May 21–29, 1919	8	73	62	.80	18.8	16.3	.96	1.76	.22	19.8	17.6	.84	1.64	.20
June 18–July 1, 1919	13	70	74	.38	18.0	13.5	1.73	2.11	.16	19.5	18.3	.46	.84	.06
July 1–July 9, 1919	8	58	78	.00	13.5	10.6	1.11	1.11	.14	18.3	16.2	.80	.80	.10
July 21–28, 1919	7	66	78	.00	15.6	13.8	.69	.69	.10	19.6	18.4	.46	.46	.06
July 28–Aug. 4, 1919	7	67	75	.86	13.8	11.3	.96	1.82	.26	18.4	17.1	.50	1.36	.19
Sept. 2–15, 1919	13	63	71	.46	17.7	12.8	1.88	2.34	.18	17.1	13.7	1.30	1.76	.14
Sept. 15–Oct. 3, 1919	18	67	69	.88	12.8	11.3	.57	1.45	.08	13.7	13.2	.19	1.07	.06

*Probably too low.
†Average for depth of from 12 to 24 inches.

ber of days, the mean relative humidity, and the mean temperature are also included for each period. The maximum rates of evaporation in inches per day have been set in bold face type. The minimum values have no significance.

It will be noticed that the rates of evaporation vary from 0.02 to 0.62 inches per day for the soil under the sod and from 0.04 to 0.68 inches per day for the soil under the bare surface. The maximum values of 0.62 and 0.68 inches occurred during the 5-day period from August 23 to 28, 1918. The value of 0.68 inches for the soil under the bare surface is probably too low, as indicated in the table. The samples taken in the upper foot of soil under the bare surface on August 28 were disturbed in drying so that the value of 6.0 per cent given for that date is the average for the second foot of depth only. In calculating the evaporation it was assumed that the percentage of moisture in the upper foot of soil on August 28 was the same as on August 23, whereas it probably was a little less.

These average values of 0.62 and 0.68 inches per day for five days are rather unusual. However, a study of the weather records shows that the conditions at that time were favorable for high rates of evaporation. The greater part of the total rainfall of 1.54 inches fell in three separate showers on three different days, August 26, 27, and 28. Considerable sunshine and some wind occurred between showers on these dates, as well as on August 23, 24, and 25.

In this connection it may be noted that Briggs and Shantz, in their experiments at Akron, Colorado, obtained values for plant transpiration, alone, which were somewhat higher than the above values.* Although, there is, of course, a great difference in climate between Dayton and Akron, it may be interesting to give some of their results. During the 10-day period from July 7 to 16, 1914, they obtained the following average daily transpiration rates in inches:

Kubanka wheat	1.21
Galgalos wheat	1.42
Swedish oats	1.31
Burt oats	1.23
Barley	0.67
Rye	0.92

*Daily Transpiration During the Normal Growth Period and Its Correlation with the Weather, by Lyman J. Briggs and H. L. Shantz, Journal of Agricultural Research, U. S. Department of Agriculture, October 23, 1916.

Cowpea	0.96
Siberian Millet	0.78
Northwestern Dent Corn	0.83
Minnesota Amber Sorghum	0.81
Sudan grass	0.66

The daily rate of evaporation from water in a shallow tank during this period was 0.48 inches. The quantities of water used by the plants were determined by weighing. Plants were grown in cans, fitted with covers to prevent soil evaporation, small holes being cut in the covers for the stems. The above transpiration rates were calculated, taking the area of the can as the area occupied by the plant. Probably a somewhat larger area should have been used, since the foliage, in some instances, undoubtedly spread out toward the light beyond the edges of the can. However, if an area twice as great as that of the can had been used, the results would still be comparatively large.

The values of evaporation given for the soil under the sod at Moraine Park, include the water intercepted by the grass and evaporated directly into the air without reaching the soil, the water taken up from the soil by the grass roots and transpired into the atmosphere, and the water evaporated directly from the surface of the soil itself. The values for the soil under the bare surface represent soil evaporation alone. While the accuracy of the data is not such as to warrant definite comparisons, it may be stated that the evaporation rate seems to be a little greater from the bare soil than from the sod for those periods in which there was considerable rainfall, and a little greater from the sod than from the bare soil for those periods in which there was no rainfall or only an insignificant amount.

The rates of soil evaporation and transpiration during the winter months were probably much lower than those given in table 6. There were probably days in the winter, during the months of January and February, when the rates were less than a hundredth of an inch per day. The observations recorded in table 3 are hardly sufficient for a discussion of minimum values. However, they do give some indication of the maximum rates which may occur during the winter months. The comparatively low rates of 0.017 for the soil under the sod and 0.049 for the soil under the bare surface, during the period from January 24 to February 11, 1919, when there was only 0.08 inches of rainfall, have already been noted. These values are probably fairly indicative of the maximum rates at which moisture can be eva-

porated from the soil at Moraine Park, during the months of January and February, when there is no appreciable precipitation. These rates would, of course, have been higher if several light rains had occurred, separated by periods of clear and windy weather. As before mentioned, the conditions at that time were about as favorable for the drying out of the soil as they ever are in the winter. These values would indicate that in the winter the evaporation from bare soil surfaces is greater than from sod surfaces, which seems reasonable.

The records taken during the winter of 1917 and 1918 offer an opportunity for estimating the rate of evaporation from snow surfaces. The greater part of the precipitation between December 3, 1917, and February 11, 1918, occurred as snow, the temperatures being below freezing the greater part of the time. The ground froze to a depth of a few inches during the cold period of December 6 to 18 when the temperature was frequently from 3 to 8 degrees below zero. It then thawed out partly and the upper foot became saturated during the period of December 21 to 29, due to the melting of about 9 inches of snow; after which it froze again and remained frozen until after the observations of February 11. It is doubtful if any appreciable amount of water percolated through the surface soil during the thawing period of December. Deducting the average runoff of 2.66 inches, from the four plats, from the rainfall of 5.26 inches leaves 2.60 inches to be accounted for by soil absorption or evaporation. The former was probably about an inch. The soil samples would indicate an average absorption of 1.65 inches, but a part of this was probably runoff from the hillside above. This leaves 1.60 inches for the total evaporation during the period of 70 days, or about 0.023 inches per day, nearly all of which must have occurred from snow surfaces. The actual daily rates undoubtedly varied a great deal from this average since there were wide variations in temperature and other meteorological conditions. The minimum, mean, and maximum values of relative humidity, temperature, and wind velocity during the 70 days were as follows:

	Minimum	Mean	Maximum
Relative humidity in per cent	51	83	100
Temperature in degrees F.	-16	19	62
Wind velocity in miles per hour	0	11.5	45

There were 18 cloudy days, 22 partly cloudy days, 30 clear days and 31 days on which the precipitation amounted to or exceeded 0.01 of an inch.

It is interesting to note that R. E. Horton* obtained an average rate of 0.028 inches per day for the period of 9 days from December 26, 1913, to January 4, 1914, at Albany, New York, when the mean maximum temperature was 26.6 degrees.

Measurements of evaporation from snow surfaces were also made in the Little Bear Valley of the San Bernardino Mountains, California, where the meterological conditions are considerably different from those at Dayton. The results, given in water supply paper 294,† were as follows:

Month and year	Average Rate in Inches per day
March, 1895	0.08
March, 1896	0.12
January, 1897	0.05
February, 1897	0.10
March, 1897	0.10

Some data on evaporation from ice surfaces in Maine is given in water supply paper 279.**

An evaporation of 0.51 inches in 6 days, with a maximum rate for one day of 0.15 inches and a minimum rate for one day of 0.03 inches, was measured at Lewiston during the period from November 19 to 24, 1905, when the average air temperature was 34.8 degrees and the average relative humidity was 39.3 per cent.

Absorption in Surface Soil

The records given in table 3 and discussed in the preceding pages enable us to study the absorption of the upper two feet of soil, or surface soil, as it may be termed. The term absorption will here be used to mean the water taken up by the soil and held to supply soil evaporation and transpiration after the rain ceases. It will not include the water that percolates through the surface soil into the underlying gravel to maintain ground water flow.

*Evaporation from Snow and Errors of Rain Gage when used to catch Snowfall, by R. E. Horton, Monthly Weather Review, February, 1914, page 99.

†An Intensive Study of the Water Resources of a Part of Owens Valley, California, by Charles H. Lee, U. S. Geological Survey Water Supply Paper 294, 1912, pages 49 and 118.

**Water Resources of the Penobscot River Basin, Maine, by H. K. Barrows and C. C. Babb, U S. Geological Survey Water Supply Paper 279, 1912, page 120.

Table 7.—Soil Absorption at Moraine Park During Summer Storms

Storm Period	Rainfall	Under Sod			Under Bare Soil		
		Moisture in Soil		Absorption*	Moisture in Soil		Absorption*
	Inches	Before Rain %	After Rain %	Inches	Before Rain %	After Rain %	Inches
Aug. 4– 8, 1916....	4.56	4.7	14.9	3.92	3.1	14.2	4.26
Sept. 5– 6, 1916....	4.12	9.3	15.4	2.34	3.7	9.0	2.03
Sept. 27–29, 1916....	1.68	8.7	16.4	2.96	7.4	11.3	1.50
Nov. 23–24, 1916....	0.84	11.3	16.9	2.15	11.7	15.1	1.30
June 26–30, 1917....	2.90	9.9	17.2	2.80	10.4	13.3	1.11
Oct. 11–19, 1917....	2.02	9.2	16.2	2.69	11.1	16.0	1.88
July 22–23, 1918....	3.11	10.3	16.4	2.34	5.8	9.0	1.23
Aug. 12–23, 1918....	2.30	5.8	15.0	3.63	6.5	15.8	3.57
Aug. 28–Sept. 2, 1918....	1.39	10.9	18.0	2.73	6.0	12.7	2.57

*In upper two feet of soil.

The preceding discussion has shown that throughout the period of about four and a half years covered by the observations the amount of moisture in the upper two feet of soil at Moraine Park varied from a minimum of 4.7 per cent, or 1.80 inches, to a maximum of 21.0 per cent, or 8.06 inches, in the case of the sod covering; and from a minimum of 3.1 per cent, or 1.19 inches, to a maximum of 21.1 per cent, or 8.10 inches, in the case of the bare soil covering; the difference in the actual amounts of water in the 2-foot layer in the two cases corresponding to 6.26 and 6.91 inches, respectively, averaging 6.58 inches. This average value would be the maximum possible absorption at Moraine Park as shown by the records. While the maximum percentages used above were practically reached each winter, the minimum values were reached only once. Consequently this average value of 6.58 inches is one which would very seldom, if ever, be attained during a single storm. For the amount of soil moisture to be increased from the minimum value to the maximum during a single storm would require an exceptional combination of conditions such as the occurrence of a very heavy prolonged rainfall at a time when the ground was dryest. Taking a value of 8 per cent for the minimum amount of soil moisture, a value which is reached practically every summer, the difference between the amount of water in the upper 2 feet during the ordinary dry periods of the summer and the amount present during the winter, would be 5.00 inches. Probably this value is also greater than the maximum amount of water ever absorbed during a single storm.

If the ground is not frozen the proportion of the maximum possible absorption that can be absorbed during a single storm varies with the amount of moisture present when the rain begins. The dryer the soil the greater is the space in which the water can be absorbed. If the rain continues long enough the moisture holding capacity of the upper two feet at Moraine Park will become filled, after which the water will percolate through the underlying sand and gravel as fast as it can move through the surface soil. If the ground is frozen very little moisture will be absorbed unless the duration of the rainfall and the temperatures are great enough to thaw out the ground. The actual amount of water that is absorbed during a given storm, of course, varies also with the nature of the rainfall. If the rainfall intensity is greater than the rate at which the water can soak into the ground, and the surface storage has been filled, the excess water will run off; whereas, if the same total precipitation is distributed through a greater time, it may all be absorbed.

Table 7 gives the larger records of absorption during individual storms, selected from the data in table 3. All records corresponding to depths of two inches or more in the upper two feet are included except in one or two instances where the data seemed questionable. In addition, records corresponding to depths of less than two inches are included for one type of surface covering where the absorption under the other type amounted to or exceeded two inches. The percentages of moisture present before the rain began and after the rain ceased, as well as the total precipitation during each storm period, are also included. The distribution of the rainfall can be seen by referring to figures 4 to 8, inclusive. The maximum values of absorption given in the table are set in bold face type. The minimum values have no special significance.

It will be noticed that in several instances the absorption was greater than the rainfall. The reason for this is that the place where the samples were taken is located near the foot of a steep hill in the direct path of the surface runoff from the hillside. As some runoff occurred during each of the storms recorded in table 7, except the one of September 27 to 29, 1916, the amount of water available to replenish the soil moisture was actually greater than the rainfall. The discrepancy in case of the storm noted is probably due to the difficulties encountered in securing samples representative of average conditions.

It is interesting to note that while runoff occurred on the

plats having the bare surface in all but one of the storms listed in table 7, and on the plats having the sod covering in the greater number of the storms, in no case did the ground become saturated. The greatest amount of moisture found in the soil after the rain ceased was only 18 per cent in the case of the sod covering, measured after the storm of August 28 to September 2, 1918; and only 16.0 per cent in the case of the bare soil, measured after the storm of October 11 to 19, 1917.

Inspection of table 7 shows that the moisture absorbed by the sod is generally greater than that absorbed by the bare soil, as would be expected. There are only two exceptions to this, and in these instances the differences are so small as to come within the limits of possible errors. In one case, that of August 4 to 8, 1916, the values constitute the maxima of the entire record, amounting to 3.92 inches for the sod and to 4.26 inches for the bare soil. This storm, with a total rainfall of 4.56 inches spread rather uniformly over five days, began when the soil was dryer than at any other time during the period covered by the data. In the other case, that of August 12 to 23, 1918, the values are the next largest, amounting to 3.53 inches for the soil under the sod and to 3.57 inches for the soil under the bare surface. The greater part of the total precipitation of 2.30 inches which fell during this period, occurred in three separate showers on three different days, August 12, 14, and 17. These are the only two instances where the absorption exceeded three inches; and it is interesting to note that both of them occurred during the month of August. All of the storm periods given in table 7 occurred in the summer or fall, during the months of June to November, inclusive. As shown in the preceding discussions, it is only during the summer months that the soil becomes dry enough to absorb such large amounts.

The values of 2.34 inches for the soil under the sod and 2.03 inches for the soil under the bare surface, obtained during the period from September 5 to 6, 1916, were caused by a total rainfall of 4.12 inches which fell in the afternoon and evening of the 5th and in the morning of the 6th. The total period of time in which the rain fell was less than 18 hours. Probably 90 per cent of the total precipitation occurred during the 6 hours from 3 to 9 p. m., on the 5th. At the Dayton Weather Bureau station the maximum intensities were 0.43 inches in 5 minutes, 0.79 inches in 10 minutes, and 1.75 inches in 30 minutes.

The values of absorption given in table 7 are believed to

be less than the amounts that actually occurred during the given storms, the reason being that while samples were always taken within a few hours or a day after the rain ceased quite frequently samples had not been taken for several days before the rain began. In calculating the values given in the table no allowances were made for the evaporation which must have occurred between the time the samples were taken and the time the rain began.

Table 8.—Soil Absorption at Moraine Park During Winter Storms

Storm Period	Rainfall	Under Sod			Under Bare Soil		
		Moisture in Soil		Absorption*	Moisture in Soil		Absorption*
	Inches	Before Rain %	After Rain %	Inches	Before Rain %	After Rain %	Inches
Dec. 12–17, 1915	1.75	15.6	17.3	0.65	16.7	16.3	–0.15
Apr. 20–22, 1916	1.08	17.7	19.8	0.80	13.7	15.3	0.61
Mar. 13–14, 1917	1.44	19.8	19.0	–0.30	17.8	20.3	0.96
Dec. 9–13, 1918	1.59	17.4	20.0	1.00	16.9	19.3	0.92
Mar. 5–10, 1919	1.09	19.9	21.0	0.42	18.0	19.6	0.61
Mar. 15–18, 1919	2.97	20.4	20.6	0.07	17.2	18.4	0.46
Mar. 26–27, 1919	0.85	18.6	20.6	0.77	17.7	19.2	0.57
Apr. 9–11, 1919	1.53	18.6	19.4	0.30	18.0	18.3	0.11

*In upper two feet of soil.

In order to show how much moisture is absorbed by the soil during winter rains a few of the larger records of absorption during individual winter storms, selected from the data in table 3, are assembled in table 8. The percentages of moisture present before the rain began and after the rain ceased and the total precipitation during each storm period are included, as in table 7. Only storms occurring during the months of December to April, inclusive, at times when the ground was not frozen, are considered. Maximum values are indicated as before.

The negative values appearing in table 8 are probably due to errors in observation as it is hardly likely that the soil was drier after the rain than it was before. It will be noticed that in only one case, that of the soil under the sod in December, 1918, was the absorption as much as an inch. There does not seem to be much difference in the winter between the absorption under the sod and under the bare soil surface. The ground was frozen during the greater number of storms which occurred in the months of January and February, and consequently such records were not included in table 8.

It will be observed that in the greater number of storms listed in table 8 the soil was nearly saturated when the rain began and was practically saturated when the rain ceased. It is interesting to point out that although the ground was saturated by the storms of March 5 to 10 and 26 to 27, 1919, no trace of runoff could be observed on any of the plats. In these instances the rates of precipitation must have been less than the rate at which the water could percolate through the two-foot layer of surface soil.

SURFACE RUNOFF

While the records in table 3 cannot be used to solve all problems connected with surface runoff, they do furnish some interesting information. They enable us to study the conditions under which surface runoff begins, the rates at which moisture can be absorbed by the soil, the relation of the total surface runoff to the total rainfall, during storm periods as well as during the year, and the amount of water that percolates through the surface soil to maintain the ground water flow of the streams. They also enable us to study the effect of variations in the slope of the ground, in the nature of the surface covering, in the amount of moisture in the soil when the rain begins, and in the character, intensity, and duration of the precipitation.

Surface Slope and Surface Covering

In order to study the variations in surface runoff caused by variations in surface slope and in surface covering, the larger runoff records of table 3 are assembled in table 9. Runoff measurements for all plats are included for all dates on which the observations show a total runoff of an inch or more on at least one of the plats. The differences in runoff due to variations in surface slope and to variations in surface covering have been calculated and are given in the last four columns of the table. The observed total rainfall is also included. Maximum values of the various quantities are set in bold face type as in preceding tables. It should be pointed out that the amounts of rainfall and runoff, here given, did not occur on the date of observation noted in the first column but occurred during the time between the date noted and the preceding date on which observations were made. An idea of the probable distribution of the runoff

Table 9.—Effect of Slope and Surface Cover on Runoff at Moraine Park

Date of Observation	Rainfall in Inches	Runoff, in Inches				Differences in Runoff, in Inches, due to			
		Sod Cover		Bare Soil		Variations in Slope		Variations in Surface Cover	
		Level	Sloping	Level	Sloping	Sod	Bare	Level	Sloping
Aug. 18, 1915....	4.22	0.61	1.33	1.61	1.73	0.72	0.12	1.00	0.40
June 3, 1916....	1.99	0	0.04	1.19	*1.11	0.04	—0.08	1.19	1.07
Aug. 7, 1916....	3.63	0	0.24	1.55	*0.97	0.24	—0.58	1.55	0.73
Sept. 6, 1916....	4.32	0	0.05	*1.90	1.66	0.05	—0.24	1.90	1.61
Jan. 8, 1917....	2.04	0	0	1.04	0.41	0	—0.63	1.04	0.41
June 11, 1917....	2.77	0.01	0	1.08	0.73	—0.01	—0.35	1.07	0.73
June 30, 1917....	2.90	0.02	0.03	*1.73	1.66	0.01	—0.07	1.71	1.63
Aug. 25, 1917....	3.04	0.03	0.01	1.35	1.52	—0.02	0.17	1.32	1.51
Feb. 11, 1918....	5.26	†2.29	†2.74	†3.05	†2.56	0.45	—0.49	0.76	—0.18
Feb. 26, 1918....	2.54	0	0.16	1.47	*0.37	0.16	—1.10	1.47	0.21
July 23, 1918....	3.11	0.03	0.11	2.23	2.20	0.08	—0.03	2.20	2.09
Sept. 26, 1918....	3.54	0	0.01	1.17	1.24	0.01	0.07	1.17	1.23
Aug. 25, 1919....	2.56	0	0.34	1.70	1.74	0.34	0.04	1.70	1.40

*Record believed to be low.
†Record uncertain due to snow accumulations.

can be obtained by referring to the rainfall distribution shown in figures 4 to 8, inclusive.

Some of the runoff measurements given in table 9 are believed to be too low or are questionable as indicated. More detailed notes on these are given at the end of table 3. Such notes are given only in cases where there was some definite reason to question the measurement. Probably several additional records for the sloping plat with the bare soil surface are too low since in several instances the runoff measured was appreciably less than that from the level plat with the same surface covering, as indicated by the minus signs in the difference column. This condition appears to have existed more often during the larger storms, such as those listed in table 9, than during the smaller ones.

In spite of these uncertainties, however, the data in table 9 offers some interesting information. It will be noticed that the differences between the runoff from the level ground and from the hillside, either on the sod or on the bare soil, are considerably less than the differences between the runoff from the sod and from the bare soil, either on the level ground or on the hillside. The maximum values of the former amount to 0.72 and 0.17 inches, respectively, for the sod and bare soil surfaces; while the maximum values of the latter amount to 2.20 and 2.09 inches, respectively, for the level and sloping locations. This would indicate that in small areas variations in surface covering have a relatively greater effect on runoff than do variations in surface slope.

The differences between the runoff from the level ground and from the hillside, in the case of the sod covering, where there is no reason to doubt the accuracy of the results, are seen to be small in all cases, especially in comparison with the total amount of the precipitation. The maximum difference of 0.72 of an inch, found in the measurements of August 18, 1915, was only about 17.1 per cent of the rainfall. It might be mentioned that this record covered two separate showers, each of which probably caused some runoff; so that the absolute value of 0.72 of an inch would be larger than that caused by either of the showers. The next largest difference was found in the measurements of February 11, 1918, amounting to 0.45 of an inch, or to about 8.6 per cent of the precipitation. In this case the runoff was caused almost entirely by melting snows at a time when the ground was saturated and was frozen to a depth of

about eight inches. For such conditions the effects of variations in surface covering, as well as in surface slope, are probably much less important than in cases where the soil can absorb a large part of the precipitation. During the storm of July 7, 1915, not included in table 9 because the runoff did not amount to an inch on any of the plats, the difference was 0.35 of an inch, or about 16.6 per cent of the total rainfall. During this storm the greater part of the rain fell in about an hour. An examination of the data in table 3 shows that out of the 95 observations in which some runoff occurred from the sod plats, in only 5 cases did the amount from the sloping plat exceed that from the level plat by as much as 0.25 of an inch, the difference generally being much less than this.

In the case of the sod covering the maximum quantity of runoff obtained from the sloping plat when there was no runoff from the level plat was 0.34 of an inch. This value was found in the observations of August 25, 1919, when the total rainfall amounted to 2.56 inches. In the case of the bare soil surface the corresponding value, which does not appear in the table, was 0.13 of an inch, this amount being measured on November 30, 1918, when the total precipitation was only 0.73 of an inch.

On the other hand the differences between the runoff from the sod and from the bare soil are seen to be comparatively large. This is true for the sloping plats as well as for the level plats, in spite of the rather low values of the runoff from the bare soil plat on the hillside. Leaving out of consideration the data for February, 1918, when the conditions were abnormal as explained above, the differences for the level ground are seen to vary from 1.00 inch, or about 24 per cent of the rainfall, on August 18, 1915, to 2.20 inches, or about 71 per cent of the rainfall, on July 23, 1918; and those for the sloping ground are seen to vary from 0.40 of an inch, or about 9 per cent of the rainfall, on August 18, 1915, to 2.09 inches, or about 70 per cent of the rainfall, on July 23, 1918, the extreme values for the two locations occurring on the same date in each case.

The fact that variations in surface runoff due to variations in surface cover are much greater than those due to variations in surface slope is also seen by referring to the runoff records platted in figures 4 to 8, inclusive. A study of these diagrams shows that while the occurrence of appreciable runoff was comparatively frequent in the case of the bare soil, it was very infrequent in the case of the sod. In fact, runoff amounting to,

or exceeding, 0.10 of an inch was observed on only 4 dates on the level sod plat as against 60 dates on the level bare soil plat; and on only 12 dates on the sloping sod plat as against 77 dates on the sloping bare soil plat. Melting snow or rain occurring at times when the ground was frozen caused the runoff on the level sod plat in 3 of the 4 cases and on the sloping sod plat in 5 of the 12 cases.

The reason that runoff seldom occurs on the sod plats, even in unusually heavy rainstorms, is that the soil is filled with roots down to the depth where the percentage of sand and gravel present is large. Consequently the soil is unusually pervious, and the water is able to percolate downward practically as fast as the rain falls, even during unusually intense downpours.

It thus appears that on the Moraine Park plats the nature of the surface covering has an important effect on the runoff conditions while the slope of the ground is relatively unimportant.

Table 10.—Surface Runoff and Soil Moisture at Moraine Park During Summer Storms

Storm Period	Rainfall in Inches	Runoff in Inches		Moisture in upper 6 inches of Soil, in %*		Ratio of Runoff from Bare Soil to Rainfall, in %
		Sod	Bare Soil	Sod	Bare Soil	
June 1– 2, 1915..	2.01	0.02	0.35	12.5	11.9	17
July 7, 1915..	2.11	0.24	0.70	11.2	14.1	33
Sept. 5, 1915..	2.15	0	0.32	14.4	13.9	15
May 3– 7, 1916..	1.81	0	0.59	15.4	13.0	33
Aug. 4– 7, 1916..	3.63	0.12	1.26	14.9	11.9	35
Sept. 5– 6, 1916..	4.12	0.02	1.78	13.9	15.1	43
May 26–28, 1917..	2.28	0.01	0.90	15.3	17.0	39
June 2– 9, 1917..	2.77	0.01	0.90	13.9	11.4	32
June 26–28, 1917..	2.86	0.02	1.70	16.4	13.5	59
July 7, 1917..	1.25	0.01	0.30	11.9	11.9	24
July 16–17, 1917..	1.43	0.01	0.40	14.8	13.4	28
July 23–26, 1917..	0.96	0.01	0.42	16.3	12.2	44
Aug. 21–22, 1917..	3.02	0.02	1.44	15.3	10.0	48
May 11–12, 1918..	2.75	0.02	0.85	19.3	19.5	31
June 6, 1918..	1.43	0	0.28	15.0	15.4	20
Aug. 12, 1918..	1.21	0.02	0.66	12.2	7.9	54
Aug. 24, 1919..	2.56	0.17	1.72	20.6	16.6	67

*After rain ceased.

It was in view of this condition that the average values of the runoff for the two types of surface covering were calculated and recorded in columns 15 and 18 of table 3. The following studies will generally be confined to these average values.

The effect of surface slope on the total amount of runoff undoubtedly becomes more important as the size of the area in-

creases, due to the longer time required for the runoff to reach the drains. A part of the water which ran off at Moraine Park probably would have been absorbed if it had been required to flow several hundred feet to the drains instead of less than seven, since total absorption varies with the length of time the surface is covered with water. This additional absorption would undoubtedly have been larger in the case of the level plats than in the case of the sloping plats, due to the effect of slope on velocity.

Surface Runoff and Soil Moisture

It was pointed out in the discussion of the amount of water absorbed by the soil during storms that surface runoff occurred at Moraine Park at times during the summer when the upper two feet of soil was not saturated. In order to study in more detail the effect of soil moisture on the surface runoff during the summer, the records of table 3 which are best suited for such a study are arranged in table 10. The records of rainfall, runoff, and amount of moisture in the upper 6 inches of soil after the rain ceased have been included for storms occurring during the months of May to September, inclusive, where the average runoff from either the sod or the bare soil amounted to or exceeded a quarter of an inch. Storms have not been included where there was any opportunity for the ground to dry out appreciably between the time the rain ceased and the time the samples were taken. The ratio of the runoff from the bare soil to the total rainfall is given in the last column. In a few cases, where observations had not been made just before and just after the given periods, the total precipitation was calculated from the Dayton records, assuming that the ratio of the total rainfall during the storm to the total observed was the same at both locations. In such cases it was also assumed that all of the runoff occurred during the storm period.

The data in table 10 shows that while a considerable proportion of the rainfall ran off of the bare soil during each of the storms included, in only one case, that of May 11 to 12, 1918, was the upper 6 inches of soil found to be nearly saturated after the rain had ceased. The samples taken after this storm showed a moisture content of 19.5 per cent for the upper 6 inches under the bare surface. The corresponding observations for the other storms included in the table are seen to vary from 7.9 per cent to 17.0 per cent, averaging about 13.0 per cent. The preceding discussion of soil moisture showed that the soil under the bare

surface can contain a quantity of water equivalent to 25.6 per cent of its dry weight, that single observations showed as much as 23 per cent for a depth of 6 inches, and that average amounts for the upper 2 feet of as much as 21 per cent were measured. It thus appears that the runoff during these storms was caused by the rain falling faster than it could soak into the ground, rather than to rain falling on a saturated soil.

The storms of August 4 to 7, 1916, September 5 to 6, 1916, June 26 to 28, 1917, August 21 to 22, 1917, and August 12, 1918, occurred at times when the ground was unusually dry and was baked hard. The other storms occurred at times when the ground was neither unusually dry nor unusually wet. It may be interesting to note that considerable flood runoff occurred throughout the Miami Valley during the storms of July 7, 1915, May 3 to 7, 1916, June 26 to 28, 1917, July 16 to 17, 1917, and May 11 to 12, 1918; also that some flood runoff was observed at a few of the gaging stations on the smaller streams of the valley during each of the other storms included in table 10.

Some interesting observations on surface runoff during intense rates of precipitation when the soil was unusually dry were made at Carrmonte, about two miles north of Moraine Park, on August 5 and 6, 1916. Rain began falling at an intense rate at 2:40 p. m. August 5. At 2:50 p. m. the precipitation, which had been fairly steady, amounted to 0.56 of an inch, corresponding to an average rate of 3.36 inches per hour for the ten minutes. At this time water was running off wherever there was enough slope in the ground surface to allow it to do so. It was running down a gravel alley which had a comparatively flat slope, and was standing all over a level lawn, running off wherever it could. Before the rain began the ground was unusually dry and was baked hard. In fact it was drier than at any other time during the period covered by the Moraine Park experiments. At 3:20 p. m., when the rain ended, the total precipitation amounted to 1.10 inches. Probably not more than half of this quantity was absorbed by the soil.

At 4:35 p. m. on August 6, rain began falling again at a fairly intense rate. At 4:50 p. m. the precipitation, which had been steady, amounted to 0.37 of an inch, corresponding to an average rate of 1.48 inches per hour for the fifteen minutes. At this time the water was running off the same areas under practically the same conditions as on the preceding day.

Interesting information regarding runoff from a small area

in Arkansas, caused by an intense shower occurring when the ground was dry and baked hard, was given by James H. Fuertes in the Journal of the Western Society of Engineers, April, 1899, page 170. An abstract of his article is given on pages 265 and 266 of Metcalf and Eddy's American Sewerage Practice, Volume I. A total runoff of 0.38 of an inch occurred from an area of about 2400 square feet, the surface of which sloped uniformly at a rate of about 5 feet in 100 feet, due to a total precipitation of 1.3 inches in 37 minutes, 0.71 of which fell in 8 minutes. The maximum rate for 5 minutes was 6 inches per hour, or a total of 0.50 inches in the 5 minutes.

It was also pointed out in the discussion of the amount of water absorbed by the soil during storms, that surface runoff

Table 11.—Winter Storms at Moraine Park Which Did Not Cause Appreciable Runoff

Sorm Period	Rainfall in Inches	Runoff in Inches		Moisture in upper 6" of Soil, in %			
				Sod		Bare Soil	
		Sod	Bare Soil	Before	After	Before	After
Dec. 16–17, 1915.....	1.50	0.02	0.03	15.6	17.1	15.5	15.6
Jan. 27–31, 1916.....	2.39	0.01	0.05	16.0	16.4	18.0	16.5
Apr. 1– 2, 1917.....	0.78	0	0	16.7	19.4	19.7	18.2
Jan. 1, 1919.....	0.75	0	0	19.4	20.0	19.9	21.1
Mar. 5– 8, 1919.....	1.09	0	0	16.8	18.2	18.0	19.0
Mar. 15–18, 1919.....	2.97	0.02	0.06	18.4	19.6	17.6	18.8
Mar. 27, 1919.....	0.85	0	0	17.1	18.1	16.1	19.2

did not occur on the plats at Moraine Park at times in the winter when the ground was apparently about as wet as it ever is. In order to show this condition more fully the more pertinent records of table 3 are brought together in table 11. Records of rainfall, runoff, and moisture in the upper 6 inches of soil before the rain began and after the rain ceased, are included for storms occurring during the months of December to April, inclusive, where the total precipitation amounted to or exceeded 0.75 of an inch, where the average runoff did not amount to as much as 0.10 of an inch from either the bare soil or the sod, and where there were no complicating conditions which would affect the results. Storms in which the precipitation occurred as snow are not included. In a few cases the Dayton daily records have been utilized in determining the total rainfall during the storm periods, as explained in the discussion of table 10.

A study of the data in table 11 shows that while in two cases the upper 6 inches of soil was not saturated when the rain ceased,

in the remaining instances the ground contained about as much moisture as it can hold under field conditions. Moreover, in the cases where the soil was not saturated, the storms of December 16 and 17, 1915, and January 27 to 31, 1916, the amount of moisture present was as great or greater than in the cases included in table 10. No appreciable runoff occurred from either the bare soil or the sod during any of the storms included in table 11 while considerable runoff occurred from the bare soil during all of the storms included in table 10. The fact that runoff did not occur from the bare soil, where there was no appreciable surface storage, during the storms included in table 11, although the ground was practically saturated in most cases, indicates that in these instances the rates of precipitation were less than the rates at which water could be absorbed or could percolate through the two-foot layer of surface soil into the underlying porous sand and gravel. Although parts of the precipitation on the sod plats, where there was some surface storage, may be accounted for by evaporation, it does not seem probable that any perceptible amount of the precipitation on the bare soil plats could be accounted for in this way. On these plats practically all of the water must have been absorbed by the soil or have percolated into the underlying materials. During the larger storms such as those of January 27 to 31, 1916, and March 15 to 18, 1919, considerable quantities of water must have percolated.

It thus appears that for bare soil, where there is practically no surface storage, intensity of precipitation has an important effect on runoff. If the intensity is greater than the rate at which water can be absorbed by the surface soil, or greater than the rate at which it can percolate through the soil when the ground is already saturated, the excess water must run off. In such cases the amount of runoff must increase as the intensity of rainfall increases. Where the soil is covered with sod or with agricultural crops the surface storage acts as a regulating reservoir, holding the water which falls during the periods when the intensity is greater than the rates at which the water can enter the ground, and allowing it to soak into the soil later, when the precipitation is less intense. Consequently in such cases, variations in the intensity of the precipitation are relatively less important as regards runoff.

Rates of Absorption and Percolation

In the case of the bare soil plats at Moraine Park a study of the intensities of precipitation during some of the storms may

furnish approximate information regarding the rates at which water can be absorbed by, or can percolate through, the two-foot layer of surface soil. The only data on intensities available for such a study are the graphical rainfall records maintained by the U. S. Weather Bureau at Dayton, about five miles north of Moraine Park. During some of the storms for which data is available, such as the local thunderstorm of August 24, 1919, where the total precipitation at the experimental plats was about four times as great as at Dayton, the intensities at the latter location were, of course, quite different from those at Moraine

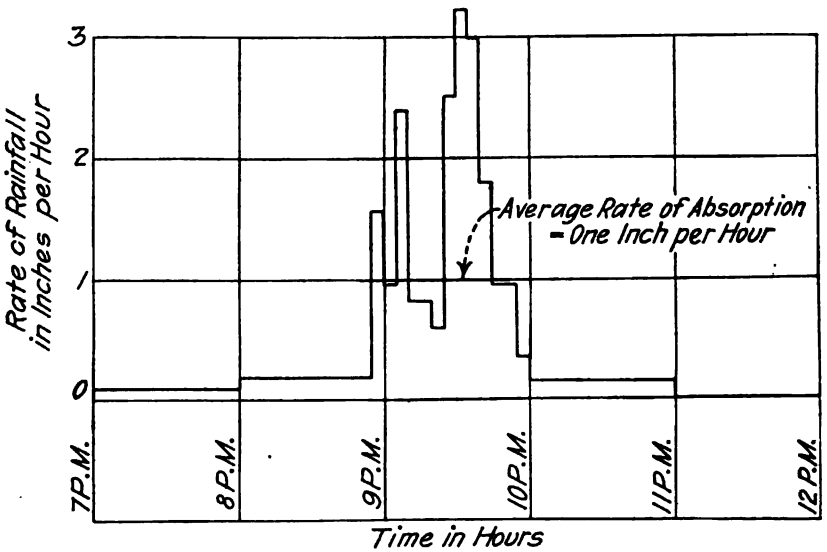


FIG. 9.—RATE OF RAINFALL AT MORAINE PARK DURING THE STORM OF JULY 7, 1915.

The average rate of absorption indicated by the horizontal line cutting through the periods of most intense precipitation, applies only to the bare soil plats.

Park. During others, however, such as the storms of July, 1915, and March 15 to 18, 1919, which were fairly uniform throughout the Miami Valley, it is believed that the Dayton records furnish fairly reliable information regarding the Moraine Park intensities.

It is also possible that a study of the excessive precipitation, as tabulated from the graphical records and published in the Monthly Weather Review, in connection with a study of the occurrence of runoff on the bare soil plats may throw some light on the capacity of the soil to absorb precipitation during intense showers. "Excessive precipitation" is a term used by the U. S.

Weather Bureau to denote intensities amounting to or exceeding the following rates:

Inches	0.25	0.30	0.35	0.40	0.45	0.50	0.55	0.60	0.65	0.70	0.80
In min.	5	10	15	20	25	30	35	40	45	50	60

Studies of rates of rainfall and of percolation or absorption have been made for some of the storms for which it is believed that the graphical records may be safely used. The rates of rainfall in inches per hour at Moraine Park were first platted as illustrated in figures 9 and 10. In determining the intensities

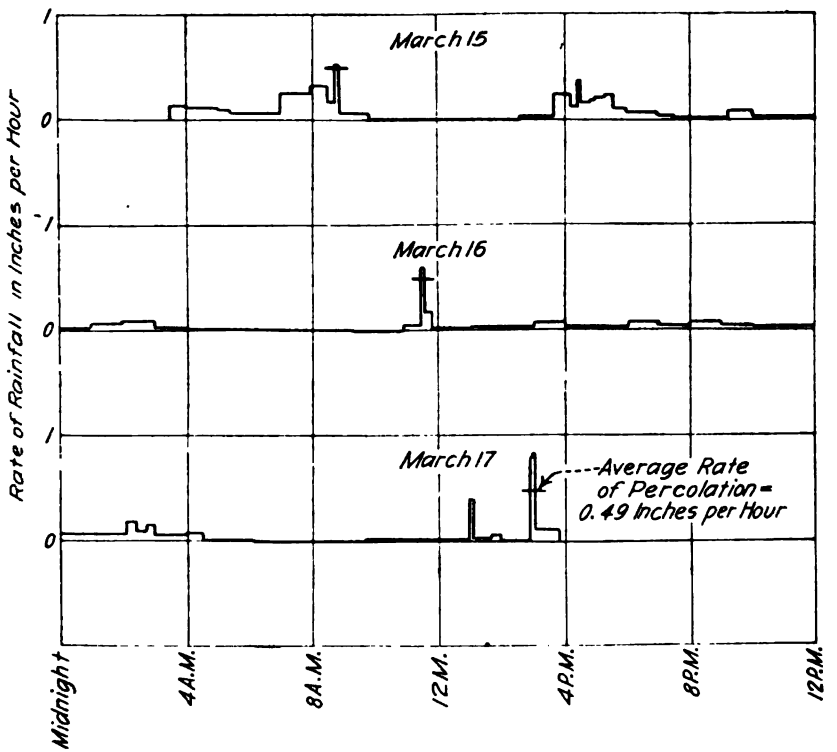


FIG. 10.—RATE OF RAINFALL AT MORAINE PARK DURING STORM OF MARCH 15 TO 17, 1919.

The average rate of percolation indicated by the horizontal line cutting through the periods of most intense precipitation, applies only to the bare soil plats.

at the plats from the Dayton records it was assumed that the rates of precipitation at the two places for short periods of time were in the same proportion as the total amounts for the storm period. While this assumption may be somewhat in error at

times, it is the only possible basis upon which to proceed. Moreover no storms have been studied where the intensities at the two locations could have been appreciably different.

The rates of precipitation were calculated for short intervals of time, by dividing the length of the interval in hours into the total rainfall in inches. In general, 5-minute intervals were used where the precipitation was excessive, and longer intervals where the rates were not so intense. Although the rates varied during the intervals used, they were platted as if they had been constant. This method is sufficiently accurate for the present purpose and has the advantage that the area under the diagram is kept exactly equal to the total precipitation from which the rates were calculated.

If it is assumed that the runoff occurred during the periods of most intense precipitation, that the amount of water evaporated during these periods was negligible, and that water entered the ground at a uniform rate at such times; then this rate of absorption, or of percolation, as the case may be, will be represented on the diagram by a horizontal line at such a height that the total area enclosed between it and the line representing the rate of precipitation will be equivalent to the total runoff. This rate would be the maximum mean rate at which water could soak into the bare soil under the conditions then existing. It would not, of course, be constant for the different storms, but would vary widely due to variations in temperature, condition of the surface, amount of soil moisture present, and rainfall intensity. In fact, it would probably vary considerably during a given storm. The assumption that it would be constant during the periods of most intense rainfall may be appreciably in error in certain cases. However, since continuous records of runoff were not secured, it is not possible to allow for such variations.

Maximum rates of absorption or of percolation were determined in the above described manner for the bare soil plats for several storms occurring during different seasons of the year. The studies for the storms of July 7, 1915, and March 15 to 17, 1919, are shown in figures 9 and 10. The rate at which the water entered the soil during the July storm has been termed "rate of absorption" since the soil was not saturated when the rain began or after it had ceased. For the March storm the rate has been termed "rate of percolation" since in this case the soil was practically saturated when the rain began, so that the

greater part of the water entering the soil must have percolated through the surface layer into the underlying sand and gravel.

The results of the studies are given in table 12. The amount of moisture in the soil under the bare surface when the rain began, the total precipitation, the total runoff, and the rates of absorption or of percolation are included for the various storms investigated. The storms are arranged in the table by dates according to the calendar year, irrespective of the year of their occurrence. The rates of percolation or absorption are the actual depths of water which would be taken up by the soil in one hour if the absorption rate existing during the period in which the runoff occurred continued that long.

A study of the data in table 12, of the rainfall distribution during the storms included in this tabulation, and of excessive rainfall records at Dayton, results in three conclusions regarding the bare soil plats at Moraine Park, as follows:

1. That water can percolate through the two-foot layer of surface soil into the underlying sand and gravel, when the ground is saturated but not frozen, at a rate as great as 0.25 of an inch per hour.

2. That water can be absorbed by the soil at times when the ground is unusually dry, at a rate as great as 1.00 inch an hour for intervals as long as 30 minutes.

3. That water cannot be absorbed by the soil at any time, no matter how dry it is, at a rate as great as 3.00 inches per hour for periods as long as 5 minutes.

Table 12.—Rainfall Intensities and Rates of Percolation and Absorption at Moraine Park

Storm	Moisture in Soil† when rain began in per cent	Total Precipitation in Inches	Total Average Runoff from Bare Soil in Inches	Average Rate* of Percolation or Absorption in Inches per hour
Mar. 15–17, 1919...	17.2	2.81	0.06	0.49
Mar. 26–27, 1916...	16.7	1.77	0.28	0.30
Apr. 9–10, 1919...	18.0	1.53	0.14	1.04
Apr. 30, 1917...	17.6	0.95	0.46	0.45
May 3– 7, 1916...	13.4	1.75	0.59	0.31
May 11–12, 1918...	15.8	2.75	0.85	0.27
July 7, 1915...	9.4	2.07	0.70	1.00
Sept. 5, 1915...	††14.3	2.15	0.32	0.40
Sept. 5– 6, 1916...	3.7	4.12	1.78	1.21

†Under bare surface.

*Depths of rainfall taken up by the soil per hour during the periods in which runoff occurred.

††Moisture content of soil on September 7.

The first conclusion was based primarily on a study of the storms of March 26 and 27, 1916, March 15 to 17, 1919, and April 30, 1917. The rate of 1.04 inches per hour, given for the storm of April 9 and 10, 1919, probably represents absorption rather than percolation. It is comparatively high due to the small amount of runoff together with the fact that the intense rates of percipitation occurred in three separate showers no one of which lasted more than five minutes.

The second conclusion was based on a study of the storms of July 7, 1915, and September 5 and 6, 1916. The records are unusually good for the former. This storm was fairly uniform throughout the valley, in intensity as well as in duration. For the latter it is believed that the absorption rate is too high due to the runoff being too low. The absorption rate for the storm of September, 1915, appears to be low although no satisfactory reason can be offered.

The third conclusion was based on a study of the excessive precipitation at Dayton, as tabulated and published by the U. S. Weather Bureau, together with the data given in table 3. It was found that some water ran off of the bare soil plats on all but one of the 31 dates on which excessive rates of precipitation occurred at Dayton. The storm in which no runoff occurred was one in which the total precipitation amounted to 0.80 inches at Dayton and only 0.07 inches at the plats, so that the intensities at the two places were not at all comparable.

Some interesting observations on rainfall intensities and runoff conditions were obtained in Dayton on April 20, 1920, when the soil was already practically saturated. A heavy shower occurring during the period from 6:30 to 6:41 a. m. caused considerable runoff from bare soil and sod surfaces, both level and sloping. The total precipitation during this time was 0.23 of an inch, corresponding to an average rate of precipitation of 1.25 inches per hour. During the period from 7:20 a. m. to 7:30 a. m., when the average rate of precipitation was 0.54 of an inch an hour, no runoff whatever could be noticed on either bare soil or sod surfaces even on comparatively steep slopes. These observations would indicate that water can be taken up by the soil, when the ground is practically saturated, at a rate as great as a half an inch an hour but that it cannot be taken up at a rate as great as an inch and a quarter an hour.

The rate at which water can enter the ground varies widely with the composition and texture of the soil, its temperature.

the type of surface covering, the amount of moisture present, and the rainfall intensity. Much has been written on the movements of soil moisture. To attempt any general discussion of the matter is beyond the scope of this publication. It may be stated, however, that the effect of initial moisture content on the rate of movement was discussed in the Journal of Agricultural Research, July 16, 1917, by F. S. Harris and H. W. Turpin.* The rate of movement was shown to increase as the initial amount of moisture present increased. It may be mentioned also that Professor King of Wisconsin University found that water would percolate through a column of clay loam 14 inches long at a rate of 1.6 inches depth in 24 hours.** However, as he did not give a mechanical analysis of the soil, it is not possible to compare his result with the rate at which water will move through the Moraine Park soil.

Conditions During Storm Periods

Table 13 contains the rainfall and runoff data for all of the large storms which occurred during the period covered by the records. The dates of the storm periods, the total precipitation, the total average runoff from each type of surface covering, the rainfall which did not run off, or the retention as it is termed, and the ratios of the runoff to the rainfall, in per cent, are included for each storm. In a few cases, where observations had not been made just before and just after the given periods, the total rainfall was calculated from the Dayton daily records, assuming that the ratio of the precipitation during the storm to the total observed was the same at both locations. This was only done in instances where it was known from a study of other data that appreciable errors could not result.

The storms are arranged in the table in groups. Those in which the precipitation occurred in one day or less, called one-day storms, are included in the first group; those in which the precipitation covered a period of more than one day and not more than two days, called two-day storms, are included in the second, and so on. In each group storms are arranged chronologically. Only those storms have been included in which the rainfall amounted to or exceeded 1.00 inch in one day, 1.50 inches

*Movement and Distribution of Moisture in the Soil, by F. S. Harris and H. W. Turpin, Journal of Agricultural Research, U. S. Department of Agriculture, July 16, 1917.

**The Soil, Its Nature, Relations, and Fundamental Principles of Management, by F. H. King, 1906, page 171.

Table 13.—Rainfall, Runoff, and Retention at Moraine Park During Storm Periods

Storm Period	Total Rainfall in Inches	Average Runoff in Inches		Retention in Inches		Ratio of Runoff to Rainfall, in %	
		Sod	Bare	Sod	Bare	Sod	Bare
ONE-DAY STORMS							
May 20, 1915	1.15	0	0	1.15	1.15	.....	.....
May 30, 1915	1.22	0.01	*0.18	1.21	1.04	0.8	14.7
June 2, 1915	1.14	0.02	0.25	1.12	0.89	1.8	21.9
June 15, 1915	1.10	0.01	0.13	1.09	0.97	0.9	11.8
July 7, 1915	2.07	0.24	0.70	1.83	1.37	11.6	33.8
Sept. 5, 1915	2.15	0	0.32	2.15	1.83	.....	14.9
June 2, 1916	1.99	0.02	*1.15	1.97	0.84	1.0	57.8
Sept. 5– 6, 1916	4.12	0.02	*1.78	4.10	2.34	0.5	43.2
Jan. 5, 1917	2.04	0	0.72	2.04	1.32	.....	35.3
Mar. 13, 1917	1.31	0.10	0.58	1.21	0.73	7.6	44.3
July 7, 1917	1.25	0.01	0.30	1.24	0.95	0.8	24.0
Oct. 18, 1917	1.29	0.01	0.54	1.28	0.75	0.8	41.9
Oct. 29, 1917	1.14	0	0.55	1.14	0.59	.....	48.2
May 12, 1918	2.23	0.02	0.85	2.21	1.38	0.9	38.1
June 6, 1918	1.32	0	0.28	1.32	1.04	.....	21.2
Aug. 12, 1918	1.21	0.02	0.66	1.19	0.55	1.7	54.5
July 20, 1919	1.66	0.01	0.10	1.65	1.56	0.6	6.0
Aug. 24, 1919	2.56	0.17	1.72	2.39	0.84	6.6	67.2
TWO-DAY STORMS							
May 20–21, 1915	1.59	0	0.04	1.59	1.55	.....	2.5
June 1– 2, 1915	2.01	0.02	0.36	1.99	1.65	1.0	17.9
Sept. 26–27, 1915	1.82	0.01	0.10	1.81	1.72	0.5	5.5
Nov. 18–19, 1915	1.55	0.01	0.01	1.54	1.54	0.6	0.6
Mar. 26–27, 1916	1.77	0.01	0.28	1.76	1.49	0.6	15.8
Sept. 27–28, 1916	1.61	0	0	1.61	1.61	.....	.....
June 27–28, 1917	2.67	0.02	*1.70	2.65	0.97	0.7	63.6
Aug. 21–22, 1917	2.95	0.02	1.44	2.93	1.51	0.7	48.8
Oct. 18–19, 1917	1.56	0.01	1.54	1.55	1.02	0.6	34.6
May 11–12, 1918	2.75	0.02	0.85	2.73	1.90	0.7	30.9
July 22–23, 1918	3.11	0.07	2.22	3.04	0.89	2.3	71.4
Apr. 9–10, 1919	1.53	0.02	0.14	1.51	1.39	1.3	9.1
May 8– 9, 1919	1.57	0.01	0.11	1.56	1.46	0.6	7.0
Oct. 26–27, 1919	2.19	0.02	0.48	2.17	1.71	0.9	21.9
THREE-DAY STORMS							
May 20–22, 1915	2.02	0.04	0.19	1.98	1.83	2.0	9.4
Jan. 10–12, 1916	2.52	0.09	0.22	2.43	2.30	3.6	8.7
Aug. 4– 6, 1916	2.51	0.12	1.26	3.39	2.25	3.4	35.9
May 26–28, 1917	2.28	0.01	0.90	2.27	1.38	0.4	39.5
July 22–24, 1918	3.68	0.07	2.33	3.61	1.35	1.9	63.3
Mar. 15–17, 1919	2.81	0.02	0.06	2.79	2.75	0.7	2.1
May 7– 9, 1919	2.19	0.01	0.33	2.18	1.86	0.5	15.1
FOUR-DAY STORMS							
Aug. 4– 7, 1916	3.63	0.12	1.26	3.51	2.37	3.3	34.7
Mar. 15–18, 1919	2.94	0.02	0.06	2.92	2.88	0.7	2.0
FIVE-DAY STORMS							
Aug. 4– 8, 1916	4.56	0.13	1.28	4.43	3.28	2.9	28.1

*Record probably low.

in two consecutive days, 2.00 inches in three consecutive days, and so on, increasing 0.50 of an inch for each additional day. Records have been included for both types of surface covering whenever the above criterion was satisfied for one type. No storms satisfying this criterion for periods longer than five days occurred during the time covered by the data. The maximum values of the various quantities are indicated as in preceding tables. Storms in which the precipitation occurred as snow have not been included.

One-day Storms

Considering the one-day storms, an examination of the data in table 13 shows that the storm of September 5 and 6, 1916, was the one in which the rainfall was greatest. While this storm is included in the one-day group, the total precipitation of 4.12 inches actually occurred in about eight hours. About 88 per cent of the total fell in four separate showers in the afternoon and evening of the fifth and on the morning of the sixth, in a total time of one and three-quarters hours. This storm occurred when the ground was unusually dry, when there was a fairly heavy growth of grass on the sod plats, and when the weather conditions were favorable for high rates of evaporation. On the sod plats, where considerable surface storage was available, only 0.02 of an inch ran off, the remaining 4.10 inches, the maximum value of the retention for the one-day storms, being taken up by the soil or evaporated between showers. This value of 4.10 inches, or practically the entire precipitation, is greater than the maximum corresponding values for the two, three, and four-day storms, being exceeded only by the value of 4.43 inches for the one five-day storm. On the bare soil plats, where there was no surface storage, 1.78 inches, or about 43 per cent of the rainfall, ran off. This is the maximum value of the runoff for the one-day periods. However, the precipitation during this storm was so great that in spite of the relatively large amount of runoff, the retention for the bare soil, 2.34 inches, or about 57 per cent of the rainfall, was not only the maximum value for the one-day storms, but was also greater than any of the two-day values.

The maximum amount of runoff from the sod plats, for the one-day periods, occurred during the storm of July 7, 1915. This was an intense storm of about an hour's duration, as shown in figure 9. The runoff amounted to 0.24 of an inch or to about 11.6 per cent of the rainfall, this value of the ratio of runoff

to rainfall also being the maximum for the one-day storms. In fact, the runoff from the sod and its ratio to the total precipitation were both greater than during any of the other storms included in table 13. The soil was neither unusually dry nor unusually wet when the rain began. The sod was covered with a fairly large growth of grass so that considerable surface storage was available.

The maximum ratio of the runoff to the rainfall for the bare soil plats, 67.2 per cent, occurred during the intense storm of August 24, 1919. This storm occurred at a time when the soil contained more than the ordinary amount of moisture.

The greater number of the one-day storms occurred during the summer and fall months. Of the eighteen storms included in this group, sixteen occurred during the months of May to November, inclusive. Only two, the storms of January 5 and March 13, 1917, occurred during the months of December to April, inclusive. During the storm of January 5, 1917, in which the rainfall amounted to 2.04 inches, there was no runoff, whatever, from the sod plats, and only 0.72 of an inch, or about 35 per cent of the rainfall, from the bare soil plats; so that the retention amounted to 2.04 and 1.32 inches, respectively for the two types of surface cover. During the storm of March 13, 1917, in which there was only 1.31 inches of rain, the runoff amounted to 0.10 of an inch, or about 7.6 per cent of the rainfall, in the case of the sod, and to 0.58 of an inch, or about 44 per cent of the rainfall, in the case of the bare soil. During this storm the values of the retention were 1.21 and 0.73 inches, respectively, for the two types of surface covering. Both of these storms occurred at times when the soil was saturated and contained no frost.

Two-day Storms

Considering the two-day storms the maximum values of the rainfall, of the runoff, and of the ratio of runoff to rainfall for both types of surface cover, and of the retention for the sod, occurred during the storm of July 22 and 23, 1918. The maximum value of retention for the bare soil occurred during the storm of May 11 and 12, 1918, amounting to 1.90 inches, or about 69 per cent of the total rainfall of 2.75 inches. During the July storm the rainfall was 3.11 inches; the runoff from the sod, 0.07 of an inch, or about 2.3 per cent of the rainfall; the retention for the sod, 3.04 inches, or about 98 per cent of the rainfall; the

runoff from the bare soil, 2.22 inches, or about 71 per cent of the rainfall; and the retention for the bare soil, 0.89 inches, or about 29 per cent of the rainfall. During the May storm the runoff from the bare soil amounted to only 0.85 of an inch, or about 31 per cent of the rainfall. The July storm occurred when the soil was unusually dry. The May storm occurred when the ground was practically saturated. The fact that the runoff from the bare soil during the July storm, when the ground was dry, was much greater than during the May storm, when the soil was practically saturated, is explained by a consideration of the intensities of the precipitation during the two storms, the intensities during the July storm being much greater than during the May storm.

It will be noticed that in this group, also, the greater number of storms occurred during the summer and fall months. Of the fourteen two-day storms, twelve occurred during the months of May to November, inclusive. Only two, the storms of March 26 and 27, 1916, and April 9 and 10, 1919, occurred during the months of December to April, inclusive. These two storms occurred when the upper foot of soil was practically saturated. During the March storm, the greater of the two, the rainfall was 1.77 inches; the runoff, 0.01 of an inch, or about 0.6 per cent of the rainfall, in the case of the sod, and 0.28 of an inch, or about 16 per cent of the rainfall, in the case of the bare soil; the retention 1.76 inches, or practically the entire rainfall, in the case of the sod, and 1.49 inches, or about 84 per cent of the rainfall, in the case of the bare soil.

Three-day Storms

In studying the three-day storms it will be noticed that the maximum values of the various quantities were distributed among four of the seven storms. The maximum values of rainfall, of runoff from the bare soil, of ratio of runoff to rainfall for the bare soil, and of retention for the sod, occurred during the storm of July 22 to 24, 1918. This storm was the one which gave the maximum two-day values. For the three-day period the rainfall amounted to 3.68 inches, a value greater than either of the storms included in the four-day group; the runoff, to 0.07 of an inch, or about 2 per cent of the rainfall, in the case of the sod, and to 2.33 inches, or about 63 per cent of the rainfall, in the case of the bare soil; and the retention to 3.61 inches, or about 98 per cent of the rainfall, in the case of the sod, and to 1.35

inches, or about 37 per cent of the rainfall in the case of the bare soil. The maximum absolute value of the runoff from the sod occurred during the storm of August 4 to 6, 1916, amounting to 0.12 of an inch, or about 3.4 per cent of the rainfall. This storm occurred when the soil was dryer than at any other time during the four and a half years covered by the data. The maximum value of the ratio of the runoff to the rainfall for the sod occurred during the storm of January 10 to 12, 1916, amounting to 3.6 per cent. The maximum value of the retention for the bare soil occurred during the storm of March 15 to 17, 1919, amounting to 2.75 inches, or to practically the entire rainfall. The comparatively small amount of runoff during this storm, only 0.06 of an inch, was due to the comparatively low rates of precipitation, see figure 10. Although the ground was practically saturated at the time, the intensities of precipitation were so low that the water could percolate through the surface soil into the sand and gravel as fast as it fell.

Only two of the seven storms included in this group fell during the months of December to April, inclusive, these two being the storms of January, 1916, and March, 1919, mentioned above. Of the remaining five, three were in May, one in July, and one in August.

Four-day Storms

Only two four-day storms appear in the table. These are the storms of August 4 to 7, 1916, and March 15 to 18, 1919, the three-day periods of which were discussed above. The retention for the bare soil during the March storm, 2.88 inches, or about 98 per cent of the rainfall, is the only quantity in this group which was not exceeded during some one of the preceding one, two, or three-day storms.

Five-day Storms

The only storm appearing in this group is the storm of August 4 to 8, 1916, the three and four-day periods of which are also included in the table. The total rainfall for the five-day period amounted to 4.56 inches. The total runoff, however, amounted to only 0.13 inches, or to about 3 per cent of the rainfall, in the case of the sod; and to only 1.28 inches, or to about 28 per cent of the rainfall in the case of the bare soil. The runoff was comparatively small due to the comparatively low rates of rainfall and due to the fact that the soil was unusually dry. The retention amounted to 4.43 inches, or about 97 per cent of

the rainfall, in the case of the sod; and to 3.28 inches, or about 72 per cent of the rainfall, in the case of the bare soil.

ANNUAL SURFACE RUNOFF

Table 14 contains the total precipitation, the total surface runoff from each type of surface cover, and the ratio of surface runoff to rainfall, for each year for which records are available. Maximum and minimum values are indicated as in preceding tables. Averages of the various quantities are given at the bottom of the table, although the four-year record is, of course, too short to give reliable averages. The quantities were calculated for the year ending September 30, in each case, since that is the year generally used in tabulating stream flow records in this part of the United States.

An examination of table 14 shows that the minimum values of annual surface runoff occurred in the year ending September 30, 1919, amounting to 0.40 inches for the sod and to 5.64 inches for the bare soil; and that the maximum values occurred in the

Table 14.—Annual Surface Runoff and Rainfall at Moraine Park

Year Ending September 30	Total Surface Runoff in Inches		Total Rainfall in Inches	Ratio of Surface Runoff to Rainfall, in per cent		
	Sod	Bare		Sod	Bare	Average
1916	.85	6.52	44.72	1.9	14.6	8.2
1917	.88	9.15	37.59	2.3	24.4	13.4
1918	2.82	12.99	38.27	7.4	34.0	20.7
1919	.40	5.64	35.77	1.1	15.8	8.4
Average	1.24	8.58	39 09	3.2	22.2	12.7

preceding year, amounting to 2.82 inches for the sod and to 12.99 inches for the bare soil. The average value of the ratio of runoff to rainfall for the two types of surface covering was 8.4 per cent in 1919 and 20.7 per cent in 1918. A value of 8. 2 per cent, or 0.2 of a per cent less than the 1919 value, occurred in 1916. The average for the four years was 12.7 per cent. The average annual surface runoff was 1.24 inches for the sod and 8.58 inches for the bare soil, averaging 4.91 inches. The average annual rainfall for the four years was 39.09 inches or slightly more than the mean annual value of 38.14 inches determined for the Dayton station from a 36-year record.

If it is assumed that the average annual runoff from the Moraine Park areas, including the ground water flow as well as

the surface runoff, is one-third of the average annual rainfall, or 13.0 inches, an assumption which will be corroborated in a later chapter, the average annual value of the ground water flow, or percolation, for the two types of surface cover, is found to be about 8.1 inches. That is, the annual surface runoff is about 12.5 per cent, and the annual percolation, about 20.7 per cent of the annual rainfall.

SUMMARY

The principal conclusions reached from the studies of the Moraine Park experiments may be summarized as follows:

1. That the surface soil, which extends only to a depth of about 2 feet, weighs about 100 pounds per cubic foot, in place, when dry.

2. That the soil when saturated contains an amount of moisture equal to about 41 per cent of the volume, or about 25 per cent of the dry weight of the soil.

3. That during the dryest times of the summer the 2-foot depth of soil never contains less than from 3 to 4 per cent of moisture, by weight.

4. That the moisture holding capacity of the 2-foot layer of soil is about 21 per cent by weight.

5. That the soil is generally dryest in the late summer or early fall, during the months of July, August, or September; and wettest in the late winter or early spring, during the months of January, February, or March.

6. That the amount of moisture in the soil gradually increases in the fall, during the months of October, November, and December; that it does not change much during the winter months, even in the absence of rainfall; and that it begins to diminish appreciably in the spring, during the months of April or May.

7. That the upper 2 feet of soil seldom, if ever, becomes filled with capillary water during the months of June, July, or August, even though the rainfall is considerably greater than normal.

8. That variations in the amount of soil moisture for short periods of time are much greater in the summer than in the winter.

9. That during the summer, rates of evaporation from bare soil and of transpiration and evaporation from sod surfaces, may be as great as a half an inch per day for periods as long as five days.

10. That during the months of January and February soil

evaporation, under the most favorable conditions, does not amount to more than from 0.02 to 0.05 of an inch per day.

11. That the average rate of evaporation from snow surfaces during the period from December 3, 1917, to February 11, 1918, was about 0.023 inches per day.

12. That the actual amount of moisture in the upper 2 feet of soil is equivalent to a depth of about 1.5 inches when the soil is dryest, and to a depth of about 8 inches when the soil contains the maximum amount of capillary water, the difference in the two amounts being about 6.5 inches.

13. That the difference between the amount of water in the upper 2 feet during the ordinary dry periods of the summer and the amount present during the winter is about 5 inches.

14. That the amount of water absorbed by the upper 2 feet during individual storms was greatest during the storm of August 4 to 8, 1916, amounting to about 4.0 inches, the total precipitation being 4.56 inches.

15. That the amount of water absorbed by the upper 2 feet during a given storm is greater under sod surfaces than under bare soil surfaces.

16. That for extremely small areas, such as the Moraine Park plats, the occurrence and amount of runoff are affected much less by surface slope than by surface cover.

17. That appreciable surface runoff frequently occurs during intense summer storms when the upper 6 inches of soil are not nearly saturated.

18. That surface runoff does not occur during some less intense storms even though the ground is saturated.

19. That water can percolate through the 2-foot layer of surface soil on the bare soil plats at Moraine Park, when the ground is saturated but not frozen, at a rate as great as 0. 25 of an inch per hour.

20. That water can be absorbed by the bare soil at times when the soil is unusually dry, at a rate as great as 1.00 inch per hour for intervals as long as 30 minutes.

21. That water cannot be absorbed by the bare soil at any time, no matter how dry it is, at a rate as great as 3.00 inches per hour for periods as long as 5 minutes.

22. That the maximum values of the retention during summer storms covering periods of 1, 2, 3, 4, and 5 days amounted to 4.10, 3.04, 3.61, 3.51, and 4.43 inches, respectively, in the case

of the sod plats, and to 2.34, 1.90, 2.25, 2.37, and 3.28 inches, respectively, in the case of the bare soil plats.

23. That similar values for winter storms for periods of 1, 2, 3, and 4 days amounted to 2.04, 1.76, 2.79, and 2.92 inches, respectively, in the case of the sod plats, and to 1.32, 1.49, 2.75, and 2.88 inches, respectively, in the case of the bare soil plats. However the values of retention for the winter storms are not directly comparable with those for the summer storms owing to differences in precipitation.

24. That the annual surface runoff at Moraine Park amounts to about one-eighth of the rainfall and that the annual percolation amounts to about one-fifth of the rainfall.

CHAPTER IV.—SPRINKLING EXPERIMENTS

In the summer of 1920 experiments on rainfall and runoff were undertaken in which rainfall effects were produced artificially by sprinkling. The work was begun at the Moraine Park plats with the object of developing a method by which rainfall and runoff relations could be determined for a given watershed, without waiting the comparatively long time required for the accumulation of sufficient records from natural rainfall. This object was accomplished in the first experiments. It was found that the sprinkling method was practicable and that the results obtained agreed with the data previously collected. It was then decided to continue the experiments at Moraine Park and also to conduct similar experiments at other places in the Miami Valley where different soil conditions existed. The object of continuing the experiments was to secure data on runoff and retention during unusually heavy rainstorms, such as those for which the Miami Valley flood prevention works are designed; and also to secure data for use in studying the general relations between rainfall and runoff.

DESCRIPTION OF PLATS

The Moraine Park plats have been fully described in the preceding chapter. Four additional plats were established at the Taylorsville Dam; two in the bottom of the valley, where the surface soil is a rich black alluvial deposit underlaid by glacial till, and two on the top of the hill near the west end of the Dam, where the surface soil is a compact yellow clay till. Two plats were established at each place so that two experiments could be run without waiting for the soil to dry out. The plats on the hill were located in an alfalfa field where the growth was comparatively thin. Those in the valley were located near the commissary gardens where a rather heavy growth of weeds had just been removed and where the soil had not been cultivated for a year or more.

The black soil in the valley extends to a depth of about two feet. For depths of from two to about eight feet the material is a yellow clay till somewhat similar to that on the hill. Below

a depth of eight feet the material is almost entirely sand and gravel. At the hill plats the yellow till extends to depths of twenty-five feet or more, and varies but little at the different depths. Some humus is present in the surface foot at each lo-

Table 15.—Mechanical Analyses of Taylorsville Soils Made by Bureau of Soils, U. S. Department of Agriculture

Sample Number	Depth below Surface Feet	Percentages of Soil particles of following diameters in mm.						
		2-1	1-0.5	0.5-0.25	0.25-0.10	0.10-0.05	0.05-0.005	0.005-0
PLATS 1 AND 2								
1	1.0	0.8	1.8	2.0	6.3	13.5	50.0	25.9
2	1.5	1.2	3.1	2.3	13.6	9.6	45.7	24.4
3	2.0	0.8	1.8	1.6	9.6	9.8	50.4	26.0
4	2.5	0.0	1.3	1.1	8.0	8.6	56.2	24.9
5	3.0	0.0	0.0	1.5	6.7	8.8	60.0	23.0
6	3.5	0.0	0.6	0.7	9.2	10.8	56.4	22.2
7	4.0	0.0	0.8	1.7	20.5	13.8	42.2	21.1
8	4.5	1.9	6.1	3.5	20.6	14.9	32.0	20.9
9	5.0	1.6	3.2	2.1	16.4	11.2	38.0	27.4
10	5.5	7.0	11.2	2.5	9.3	13.2	33.7	23.0
PLATS 3 AND 4								
11	0.5	1.0	0.8	1.1	7.0	8.3	50.2	31.5
12	1.0	1.1	3.0	1.9	11.4	9.5	38.0	35.0
13	1.5	0.8	3.0	2.2	14.5	13.5	34.2	31.8
14	2.0	1.8	4.4	2.5	12.2	14.2	38.2	26.9
15	2.5	2.8	7.1	3.5	13.3	15.3	38.0	19.9
16	3 0	2.5	6.1	3.6	13.3	14.3	42.3	18.2
17	3.5	3.1	6.2	3.6	13.2	14.6	39.4	20.2
18	4.0	3.5	6.2	3.4	13.9	13.8	39.1	20.5
19	4.5	4.3	7.0	3.7	14.0	14.6	39.2	17.0
20	5.0	3 4	6.4	3.5	13.8	15.3	38.2	19.5

cation. Table 15 gives the results of soil analyses made by the Bureau of Soils, U. S. Department of Agriculture.

In the valley the ground surface is practically level, the slopes being only great enough to cause the water to drain toward the tile outlets. On the hill the surface slopes toward the outlets at a rate of about 0.5 feet in 10 feet.

Figure 11 shows the valley plats and figure 12 shows the hill plats, plats being numbered consecutively for convenience in the following discussions. Each plat is 5 feet square, as at Moraine Park. It will be noticed that some vegetation is present in plat 1, in the valley, and in both plats on the hill. Plat 2, in the valley, was spaded to a depth of about 6 inches and then raked so as to correspond to a corn field after planting and dragging.

The plats were separated from the adjacent land by galvanized sheet iron boundaries extending about 4 inches above the

FIG. 11.—EXPERIMENTAL PLATS 1 AND 2 AT THE TAYLORSVILLE DAM.

These plats are located in the bottom of the valley where the surface soil is a rich black alluvial deposit. Note the sprinkling cans used in reproducing rainfall effects.

ground and about 20 inches below. The corners were soldered so as to be water-tight. In setting the boundaries care was taken not to disturb the areas within the plats, the trenches being dug entirely on the outside of the plats and with vertical smooth walls on the sides where the sheet iron was to be set. After setting the boundaries to the required depth the trenches were carefully backfilled and puddled so as to be as nearly water-tight as possible.

Tanks to catch the runoff were set between the plats at each location, as shown in the figures, and were connected to the plats by 3-inch sewer pipe. Care was taken to obtain water-tight joints where the pipe pass through the boundaries. One tank was sufficient for both plats in each case, since only one plat was to be experimented on at a time. The tanks were provided with covers so that they could be kept covered when experiments were not being made.

METHODS OF EXPERIMENTATION

Water was applied with the two garden sprinkling cans shown in figure 11, in all experiments except the first, when only one can was available. The cans are identical in capacity as well as in size and number of nozzle openings. In running the experiments they were filled to marks previously located, at which the capacity of the can is equivalent to a depth of 0.20 of an inch over one of the plats. The cans were marked "A" and "B" for convenience in recording the data. The amount of precipitation was determined by recording the number of cans applied, and the rate of precipitation was obtained by recording the time required to empty them. Different rates of precipitation were obtained by plugging different numbers of nozzle openings.

The amount of runoff was determined by measuring the depth of water in the tank, and the rate of runoff was obtained by noting the time required for the tank to fill to the different depths. The gage used in measuring the depths is shown in figure 11. It was made by fastening two rain gage measuring sticks on to a small rod. Depths were measured to the nearest tenth of an inch and observations were made every three or four minutes.

In running experiments one man sprinkled while another filled the empty can, measured depths in the tank, and recorded the necessary data. The general procedure was to apply water at a constant rate until the surface became saturated, and then

to vary the rate of application through a range corresponding to actual rainfall conditions. The plat was then allowed to dry out to its original condition before a second experiment was run.

A split-second stop watch was used in determining time. Although it gave a little greater precision than was necessary, it was more convenient to use than an ordinary watch. Times were observed and recorded to the nearest second.

Soil samples, for determining the amount of moisture in the ground when the experiments were started, were taken from places just outside the plats. Samples were not taken after the experiments were finished since it would have been necessary to take them from within the plats. It was desired to keep the soil within the plats undisturbed so that subsequent experiments could be made with similar soil conditions. Experiments were not repeated on a given plat until a sufficient time had elapsed for the moisture content of the soil within the plat to become equal to that of the soil just outside.

In addition to the rainfall and runoff experiments conducted on the plats, a few supplementary experiments were made on areas lying near the plats, for the purpose of determining how long different rates of rainfall must continue before runoff begins. These experiments were made by marking off areas similar in size and in surface characteristics to the areas within the plats and then sprinkling them at different rates until runoff began. The time elapsing between the beginning of the sprinkling and the beginning of the runoff was observed for each rate of application.

RESULTS OF EXPERIMENTS

The results of the different experiments are given in tables 16 to 19, inclusive. Tables 16 and 17 give the rainfall, retention, and runoff data obtained at the Moraine Park and Taylorsville plats, respectively, arranged by runs; table 18 gives the total rainfall, runoff, and retention for the various experiments; and table 19 gives the data on the conditions necessary for runoff to begin, obtained during a few special runs made at various locations.

In tables 16 and 17 the total sprinkling time in hours; the total rainfall, retention, and runoff, in inches; and the average rates of rainfall, retention, and runoff, in inches per hour, are given for the various runs in which the experiments were conducted. The total quantities for each experiment, or series of

FIG. 12.—EXPERIMENTAL PLATS 3 AND 4 AT THE TAYLORSVILLE DAM.

These plats are located on the top of the hill at the west end of the dam where the soil is a compact yellow clay till.

runs, are given immediately following the last run of the series. Columns are also included giving the experiment number, the run number, the date, the time between successive runs on the same day, or on consecutive days, in hours and minutes, the ratio of total runoff to total rainfall in per cent, the average rate of sprinkling in inches per hour before runoff began, the time in minutes before runoff began, the total precipitation before runoff began, and the condition of the plats when the experiments were started.

For runs 1 to 8, inclusive, and runs 23 and 24, given in table 16, where the water was applied with one sprinkling can, the time of sprinkling is the net time, not including the intervals required to fill the can. The rate of runoff was computed by using the total net time in which runoff occurred, and the rate of retention was obtained by assuming the time in which the retention took place to be the same as the total sprinkling time. Consequently the differences between the rates of precipitation and runoff are not exactly equal to the rates of retention. It might have been better to have computed the rates of runoff and retention in a slightly different manner but since these experiments were more or less of a preliminary nature the data has not been recalculated.

The sprinkling was continuous during all of the remaining runs of table 16 and during all runs of table 17. For these runs slightly different methods of compilation were used. In calculating the average rate of runoff for a given run the short periods of time at the beginning and ending of the run, in which the rate of runoff was changing greatly, were not considered. The rate of retention was obtained by simply subtracting the rate of runoff from the rate of rainfall.

In all instances the total time given for a certain experiment, or series of runs, is the actual time from the beginning of the first run to the end of the last run, including nights as well as other intervening periods, rather than the total of the sprinkling times for the runs included in the experiment. For runs 1 to 8, inclusive, in table 16, a second set of totals is given, in which are included the actual rainfall and runoff quantities which occurred during the evening of June 2.

In table 18 the total quantities for the different experiments are brought together so that they may be studied collectively. The total time from the beginning of the first run to the end of the last run in hours, the total sprinkling time in hours, the total

Table 16.—Results of Sprinkling Experiments at Moraine Park during 1920

Experiment	Run Number	Date	*Time between Runs in Hours and Minutes	Total Quantities: Sprinkling Time in Inches	Total Quantities: Rainfall in Inches	Total Quantities: Retention in Inches	Total Quantities: Runoff in Inches	Ratio of Runoff to Rainfall in per cent	Average Rates: Rainfall in Inches per Hour	Average Rates: Retention in Inches per Hour	Average Rates: Runoff in Inches per Hour	Before Runoff Began: Average Rate of Sprinkling in Inches per Hour	Before Runoff Began: Time in Minutes	Before Runoff Began: Total Precipitation in Inches	Remarks
LEVEL BARE SOIL PLAT															
Experiment 1	1	6- 1	.....	0.84	3.41	1.70	1.71	50	4.06	2.03	1.86	4.25	2.0	.14	Soil dry and loose when Run 1 was begun; moisture content probably 8 to 10 per cent. Sprinkling intermittent.
	2	6- 2	21-25	0.90	1.89	0.90	0.99	52	2.10	1.00	1.11	2.24	3.3	.12	
	3	6- 2	0-13	0.44	0.49	0.30	0.19	39	1.12	0.70	0.46	1.31	1.8	.04	
	4	6- 2	0- 6	0.21	0.17	0.12	0.05	29	0.81	0.56	0.44	0.81	4.2	.06	
	5	6- 3	18-15	0.86	0.66	0.38	0.28	42	0.77	0.44	0.36	0.88	4 5	.07	
	6	6- 3	0-49	0.68	0.19	0.18	0.01	5	0.27	0.26	0.04	0.27	29.9	.13	
	7	6- 3	0	0.31	0.06	0.06	0	.....	0.18	0.18	0	No	Runoff		
	8	6- 3	0-10	0.26	0.82	0.25	0.57	70	3.11	0.93	1.75	3.27	0.6	.03	
	1-8	.....	.....	†46.7	7.69	3.89	3.80	49	.....	.....	.....	.....	.....	.....	
	‡1-8	.....	.....	.....	8.70	4.26	4.44	51	.....	.....	.....	.....	.....	.....	
Experiment 3	9	7-22	.....	1.10	3.60	0.95	2.65	74	3.26	0.70	2.56	3.65	5.5	.34	Soil trampled and packed by cattle after Run 8; dry and hard; moisture content probably about 8 to 10 per cent when Run 9 was begun. Sprinkling continuous.
	10	7-22	0-11	0.96	3.20	0.41	2.79	87	3.33	0.35	2.98	3.27	1.1	.06	
	11	7-22	1-58	0.91	3.20	0.35	2.85	89	3.54	0.38	3.16	3.40	1.5	.09	
	12	7-23	18-57	1.76	3.00	0.46	2.54	85	1.70	0.13	1.57	1.76	9.5	.28	
	13	7-23	1-22	1.35	1.20	0.22	0.98	82	0.89	0.12	0.77	0.94	4.7	.07	
	14	7-23	0- 1	0.74	0.40	0.05	0.35	87	0.54	0.06	0.48	.....	0	0	
	15	7-23	0- 1	0.76	0.20	0.06	0.14	70	0.26	0.09	0.17	.....	0	0	
	9-15	.....	.....	†30.7	14.80	2.50	12.30	83	.....	.....	.....	.....	.....	.....	

Table 16.—*Continued*

Experiment	Run Number	Date	*Time between Runs in Hours and Minutes	Total Quantities: Sprinkling Time in Hours	Total Quantities: Rainfall in Inches	Total Quantities: Retention in Inches	Total Quantities: Runoff in Inches	Ratio of Runoff to Rainfall in per cent	Average Rates: Rainfall in Inches per Hour	Average Rates: Retention in Inches per Hour	Average Rates: Runoff in Inches per Hour	Before Runoff Began: Average Rate of Sprinkling in Inches per Hour	Before Runoff Began: Time in Minutes	Before Runoff Began: Total Precipitation in Inches	Remarks
SLOPING BARE SOIL PLAT															
Experiment 4	16	7–28	.....	1.25	4.00	1.31	2.69	67	3.20	0.90	2.30	3.00	3.5	.18	Soil dry and loose; moisture content probably 8 to 10 per cent when Run 16 was begun. Sprinkling continuous.
	17	7–28	1–59	1.05	3.60	0.84	2.76	77	3.41	0.72	2.69	3.27	0.9	.05	
	18	7–28	0–10	1.00	3.40	0.67	2.73	80	3.41	0.62	2.79	3.43	0.8	.05	
	19	7–29	17–20	2.18	4.00	1.04	2.96	74	1.84	0.46	1.38	2.18	2.2	.08	
	20	7–29	2– 1	1.32	1.20	0.48	0.72	60	0.91	0.33	0.58	0.92	2.0	.03	
	21	7–29	0– 1	0.72	0.40	0.21	0.19	48	0.55	0.28	0.27	.....	0	0	
	22	7–29	0	0.73	0.20	0.18	0.02	10	0.28	0.23	0.05	0.28	4.7	.02	
	16–22	.....	.....	†29.9	16.80	4.73	12.07	72	.....	.....	.....	.....	.....	.....	
LEVEL SOD PLAT															
Experiment 2	23	6– 3	.....	0.49	2.06	2.06	0	.....	4.17	4.17	0	No Run Off			Soil dry and very loose; full of roots. Sprinkling intermittent during Runs 23 and 24.
	24	6– 3	0– 2	0.43	6.20	6.08	0.12	2	14.4	14.1	0.4	14.4	§42.2	§4.87	
	25	6– 3	0–18	0.17	3.40	2.23	1.17	34	20.4	13.0	7.4	20.4	1.5	0.53	
	23–25	.....	.....	†2.0	11.66	‡0.37	1.29	11	.....	.....	.....	.....	.....	.....	

§Includes amounts in Run 23.
*From end of preceding run to beginning of present run.
†Total time from beginning of first run to end of last run.
‡Including actual rainfall and runoff during evening of June 2.

Table 17.—Results of Sprinkling Experiments at Taylorsville Dam during 1920

Experiment	Run Number	Date	*Time between Runs in Hours and Minutes	Total Quantities: Sprinkling Time in Hours	Total Quantities: Rainfall in Inches	Total Quantities: Retention in Inches	Total Quantities: Runoff in Inches	Ratio of Runoff to Rainfall in per cent	Average Rates: Rainfall in Inches per Hour	Average Rates: Retention in Inches per Hour	Average Rates: Runoff in Inches per Hour	Before Runoff Began: Average Sprinkling Rate Inches per Hour	Before Runoff Began: Time in Minutes	Before Runoff Began: Total Precipitation in Inches	Remarks**
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
PLAT 1															
Experiment 5	1	8– 9	.....	1.28	4.00	1.50	2.50	62	3.12	1.13	1.99	3.50	3.0	0.18	Soil dry and hard; moisture content probably about 10 per cent, when Run 1 was begun.
	2	8– 9	2–42	0.94	3.00	0.44	2.56	85	3.18	0.43	2.75	3.27	1.5	0.08	
	3	8– 9	0–16	0.99	3.00	0.36	2.64	88	3.02	0.33	2.69	3.35	1 0	0.06	
	4	8–10	17–09	1.78	3.00	0.93	2.07	69	1.69	0.50	1.19	1.64	4.0	0.11	
	5	8–10	1–38	1.36	1.20	0.49	0.71	59	0.88	0.32	0.56	0.84	4.5	0.06	
	6	8–10	0	0.77	0.40	0.24	0.16	40	0.52	0.31	0.21	.....	0	0	
	7	8–10	0	0.72	0.20	0.19	0.01	5	0.28	0.26	0.02	.....	0	0	
	1– 7	.....	.....	†29.9	14.80	4.15	10.65	72	.....	.....	.....	.....	.....	.....	
Experiment 9	8	10–19	.....	1.74	6.00	2.25	3.75	62	3.46	1.07	2.39	3.69	7.0	0.43	Soil unusually dry and hard; moisture content about 7 per cent; a few cracks in surface within plat and along boundaries, when Run 8 was begun.
	9	10–19	1–53	0.93	3.00	0.64	2.36	79	3.23	0.55	2.68	3.00	2.5	0.12	
	10	10–19	0– 9	0.88	3.00	0.46	2.54	85	3.40	0.47	2.93	3.27	0.5	0.03	
	8–10	.....	.....	†5.7	12.00	3.35	8.65	72	.....	.....	.....	.....	.....	.....	
PLAT 2															
Experiment 6	11	8–11	.....	1.88	7.00	4.77	2.23	32	3.73	2.24	1.49	3.90	22.0	1.43	Soil dry and loose; spaded to a depth of 6 inches and thoroughly raked; moisture content about 10 per cent, when Run 11 was begun.
	12	8–11	2–29	1.38	5.00	2.05	2.95	59	3.63	1.44	2.19	3.69	1.5	0.09	
	13	8–12	18–57	2.14	4.00	1.94	2.06	52	1.87	0.86	1.01	1.85	6.0	0.18	
	14	8–12	1–24	1.15	1.20	0.63	0.57	48	1.04	0.53	0.51	1.09	3.0	0.05	
	15	8–12	0– 1	0.69	0.40	0.28	0.12	30	0.58	0.39	0.19	0.57	5.0	0.05	
	16	8–12	0	0.57	0.23	0.21	0.02	9	0.41	0.35	0.06	.....	.....	.....	
	11–16	.....	.....	30.8†	17.88	9.88	7.95	45	.....	.....	.....	.....	.....	.....	

Table 17.—*Continued*

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
								PLAT 3							
Experiment 7	17	8–16	.....	0.92	3.20	0.47	2.73	85	3.49	0.39	3.10	3.43	2.5	0.14	Soil dry and hard; moisture content about 12 per cent, when Run 17 was begun.
	18	8–16	2–29	0.99	3.40	0.28	3.12	92	3.43	0.21	3.22	3.43	2.0	0.11	
	19	8–16	0–15	1.01	3.40	0.18	3.22	95	3.36	0.17	3.19	3.43	1.0	0.06	
	20	8–17	17–49	2.11	4.00	0.66	3.34	84	1.90	0.29	1.61	1.92	2.5	0.08	
	21	8–17	1–38	1.19	1.20	0.28	0.92	77	1.01	0.21	0.80	1.04	2.5	0.04	
	22	8–17	0– 2	0.69	0.40	0.12	0.28	70	0.58	0.16	0.42	.....	0	0	
	23	8–17	0	0.65	0.20	0.11	0.09	45	0.31	0.17	0.14	.....	0	0	
	17–23	.....	.....	30.4†	15.80	2.10	13.70	87	.....	.....	.....	.....	.....	.....	
Experiment 11	24	10–22	.....	1.31	2.00	1.30	0.70	35	1.53	0.95	0.58	1.43	17.5	0.30	Soil unusually dry and hard; moisture content about 7 per cent; a few cracks in surface within plat when Run 24 was begun.
	25	10–22	1–12	1.32	2.00	1.00	1.00	50	1.51	0.67	0.84	1.55	3 0	0.08	
	26	10–22	.....	1.28	2.00	0.64	1.36	68	1.56	0.54	1.02	.....	0	0	
	24–26	.....	.....	5.2 †	6.00	2.94	3.06	51	.....	.....	.....	.....	.....	.....	
								PLAT 4							
Experiment 8	27	8–18	.....	1.08	3.60	0.68	2.92	81	3.32	0.55	2.77	3.43	2.5	0.14	Soil dry and hard; moisture content about 12 per cent, when Run 27 was begun.
	28	8–18	0–17	0.94	3.20	0.37	2.83	88	3.40	0.37	3.03	3.43	1.0	0.06	
	29	8–18	1–45	0.95	3.20	0.37	2.83	88	3.38	0.34	3.04	3.35	1.0	0.05	
	30	8–19	19– 0	1.91	3.60	0.81	2.79	78	1.90	0.41	1.49	2.00	2.5	0.08	
	31	8–19	1–54	1.23	1.20	0.33	0.87	72	0.97	0.25	0.72	0.99	2.0	0.03	
	32	8–19	0	0.62	0.40	0.13	0.27	67	0.64	0.22	0.42	.....	0	.....	
	33	8–19	0	0.49	0.20	0.10	0.10	50	0.41	0.20	0.20	.....	0	.....	
	27–33	.....	.....	30.6†	15.40	2.79	12.61	82	.....	.....	.....	.....	.....	.....	
Experiment 10	34	10–21	.....	1.33	4.80	1.56	3.24	68	3.60	0.99	2.61	3.00	5.0	0.25	Soil unusually dry and hard; moisture content about 7 per cent; a few cracks in surface within plat when run 34 was begun.
	35	10–21	1–03	0.96	3.60	0.72	2.88	80	3.73	0.70	3.03	3.69	1.5	0.09	
	36	10–21	0–12	0.93	3.60	0.48	3.12	87	3.89	0.47	3.42	3.79	1.0	0.06	
	34–36	.....	.....	4.6 †	12.00	2.76	9.24	77	.....	.....	.....	.....	.....	.....	

*From end of preceding run to beginning of present run **Sprinkling continuous during all runs.
†Total time from beginning of first run to end of last run.

Table 18.—Summary of Rainfall, Retention, and Runoff during Sprinkling Experiments made in 1920

Experiment Number	Plat	Date	Total† Time in Hours	Total Sprinkling Time in Hours	Total Quantities			Ratio of Runoff to Rainfall in per cent	Ratio of Retention for Last Run in Inches per Hour
					Rainfall in Inches	Retention in Inches	Runoff in Inches		
			MORAINE PARK PLATS						
1	Level Bare Soil	June 1–3	46.7	4.50	8.70	4.26	4.44	51	0.18
2	Level Sod	June 3	2.0	1.09	11.66	10.37	1.29	11	13.0
3	Level Bare Soil	July 22–23	30.7	7.58	14.80	2.50	12.30	83	0.09
4	Sloping Bare Soil	July 28–29	29.9	8.25	16.80	4.73	12.07	72	0.23
			TAYLORSVILLE PLATS						
5	Plat 1	Aug. 9–10	29.9	7.84	14.80	4.15	10.65	72	0.26
6	Plat 2	Aug. 11–12	30.8	7.81	17.83	9.88	7.95	45	0.35
7	Plat 3	Aug. 16–17	30.4	7.56	15.80	2.10	13.70	87	0.17
8	Plat 4	Aug. 18–19	30.6	7.22	15.40	2.79	12.61	82	0.20
9	Plat 1	Oct. 19	5.7	3.55	12.00	3.35	8.65	72	0.47
10	Plat 4	Oct. 21	4.6	3.22	12.00	2.76	9.24	77	0.47
11	Plat 3	Oct. 22	5.2	3.91	6.00	2.94	3.06	51	0.54

†Total time from beginning of first run to end of last run.

Table 19.—Intensity and Duration of Precipitation Before Runoff Begins

Run Number	Location	Date	Before Runoff Began			Surface and Soil Conditions
			Average Rate of Sprinkling in Inches per Hour	Time in Minutes	Total Precipitation in Inches	
1	Moraine Park.....	June 1	3.65	2.5	0.15	Same as level bare soil plat, dry.
2	Taylorsville.....	Oct. 23	2.80	6.0	0.28	Same as plats 3 and 4, very dry.
3	Taylorsville.....	Oct. 23	2.20	10.0	0.37	Same as plats 3 and 4, very dry.
4	Taylorsville.....	Oct. 23	0.86	42.0	0.60	Same as plats 3 and 4, except a little more vegetation.
5	Taylorsville.....	Oct. 23	3.69	2.8	0.17	Made in plat 4, ground wet.
6	Taylorsville.....	Oct. 23	3.53	2.2	0.13	Made in plat 3, ground wet.

rainfall, retention, and runoff, in inches, the ratio of total runoff to total rainfall in per cent, and the rate of retention during the last run in inches per hour, are given for each experiment, experiments being arranged chronologically and numbered consecutively.

In table 19 the average rate of sprinkling in inches per hour, the time in minutes, and the total precipitation in inches, before runoff began, are given for the special runs made in order to study the conditions causing runoff to begin. Run numbers, dates, locations, and surface conditions are also noted.

Figures 13 to 21, inclusive, show graphically the data obtained in the various experiments. Figures 13 to 15 show the results obtained on the bare soil plats at Moraine Park, and figures 16 to 21 show those obtained on the Taylorsville plats. The data secured on the sod plat at Moraine Park is so unusual, as will be explained later, that it has not been platted. The rainfall, retention, and runoff are shown by means of mass curves, separate curves being drawn for each day's observations. The rates of rainfall, retention, and runoff are, of course, shown by the slopes of the lines. The various runs are numbered, as in tables 16 and 17. Intervals between runs made on the same day are indicated by the horizontal portions of the curves. The comparatively steep parts of the retention curves at the beginnings of the runs, and also the small peaks or humps near the ends of the runs, are due to surface storage.

RAINFALL, RETENTION, AND RUNOFF

Reference to table 18 shows that with one or two exceptions the total quantities of water applied in the various experiments were larger than any actual rainfalls on record in, or near, the Miami Valley. With the exception of experiments 1, 2, and 11, the rainfall varied from 12.00 inches in about 5 hours to 17.83 inches in about 30 hours; the total runoff, from 7.95 to 13.70 inches; and the total retention, from 2.10 to 9.88 inches. In experiment 1 the total rainfall was 8.70 inches in about 2 days. Experiment 2 was run on the level sod plat at Moraine Park where the soil conditions were unusual. In experiment 11 the rate of application was purposely kept low in order to study the retention for less intense storms.

As noted in tables 16 and 17 the soil in all instances, was comparatively dry when the experiments were started. The ground at Taylorsville was a little drier in October, when experi-

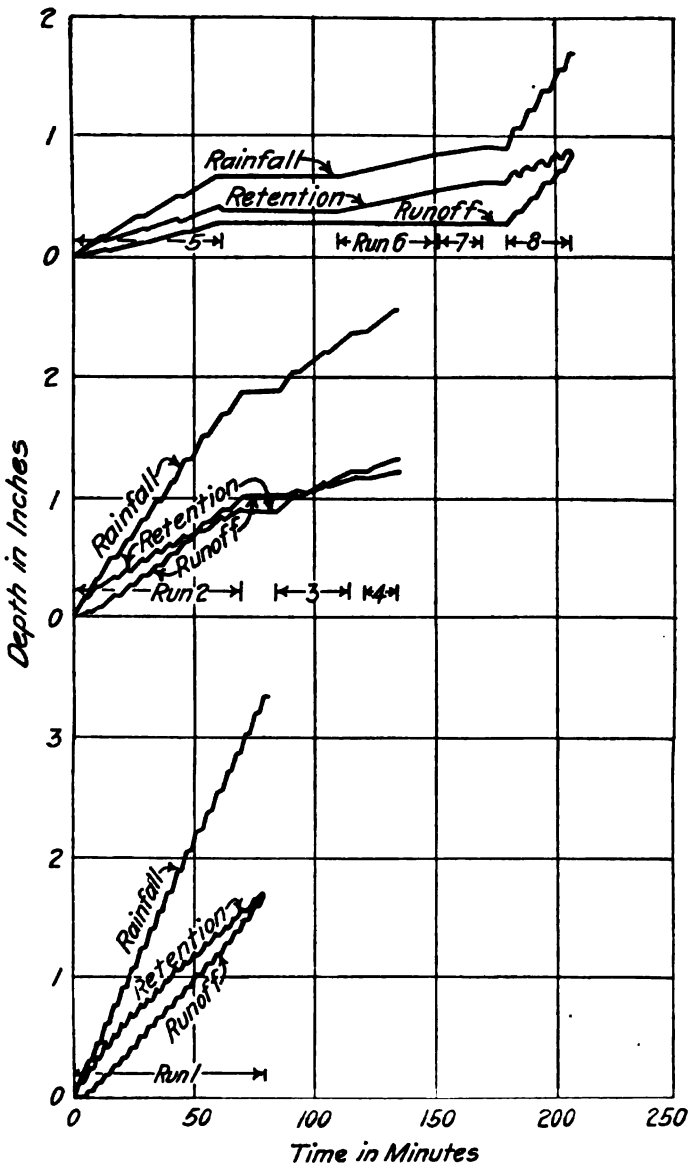


FIG. 13.—EXPERIMENT 1, MADE ON THE LEVEL BARE SOIL PLAT AT MORAINE PARK, JUNE 1 TO 3, 1920.

The curves show the total rainfall, runoff, and retention up to any instant.

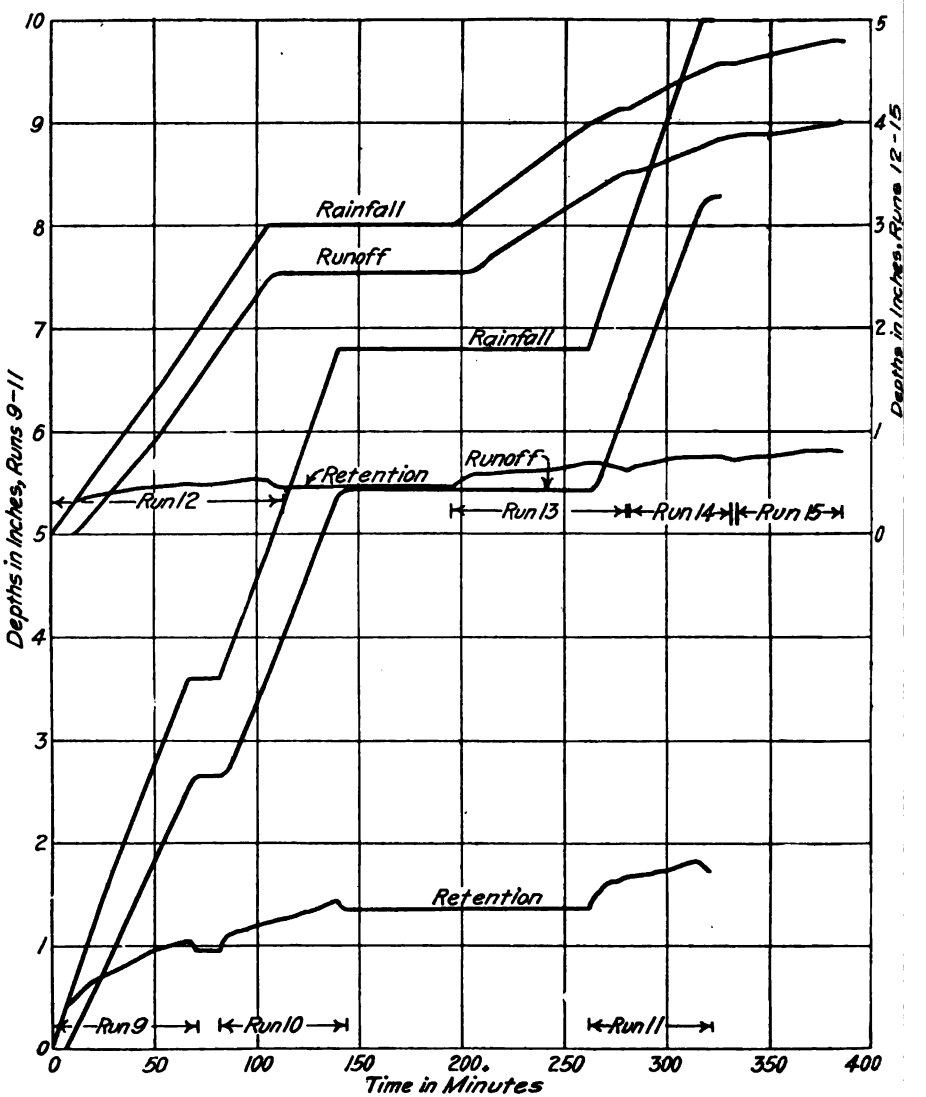


FIG. 14.—EXPERIMENT 3, MADE ON THE LEVEL BARE SOIL PLAT AT MORAINE PARK, JULY 22 TO 23, 1920.

The curves show the total rainfall, runoff, and retention up to any instant.

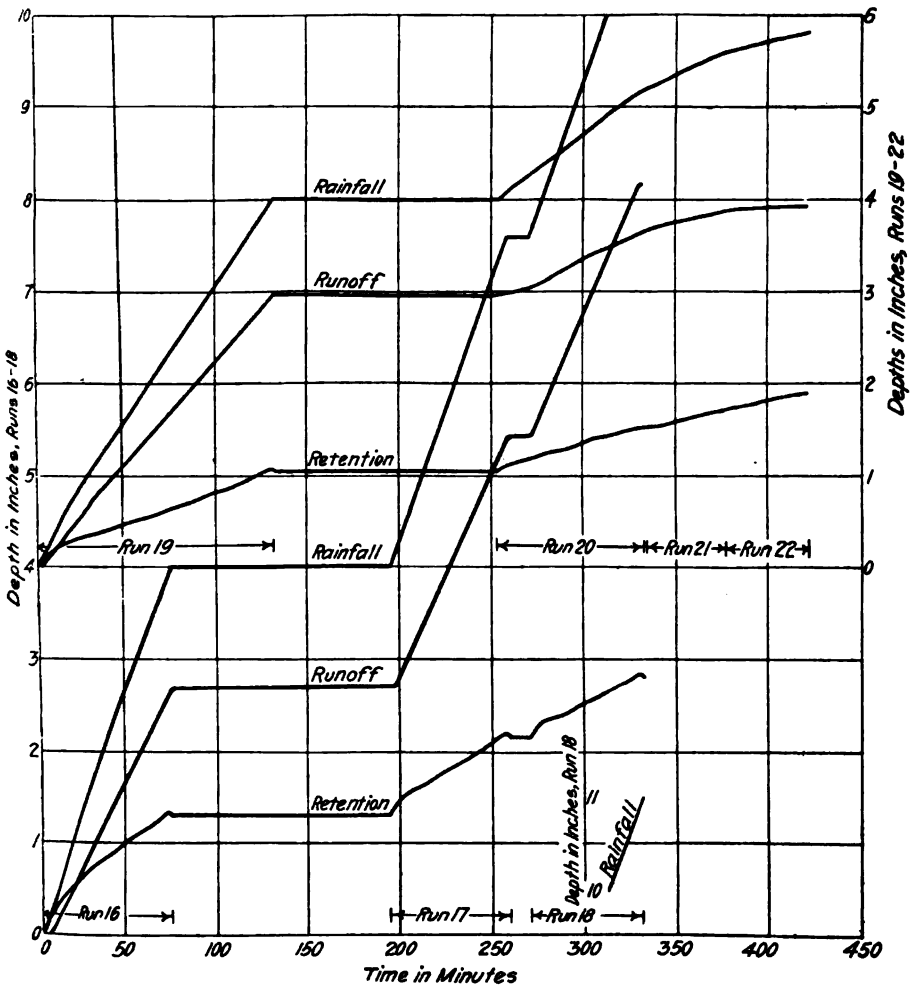


FIG. 15.—EXPERIMENT 4, MADE ON THE SLOPING BARE SOIL PLAT AT MORAINE PARK, JULY 28 TO 29, 1920.

The curves show the total rainfall, runoff, and retention up to any instant.

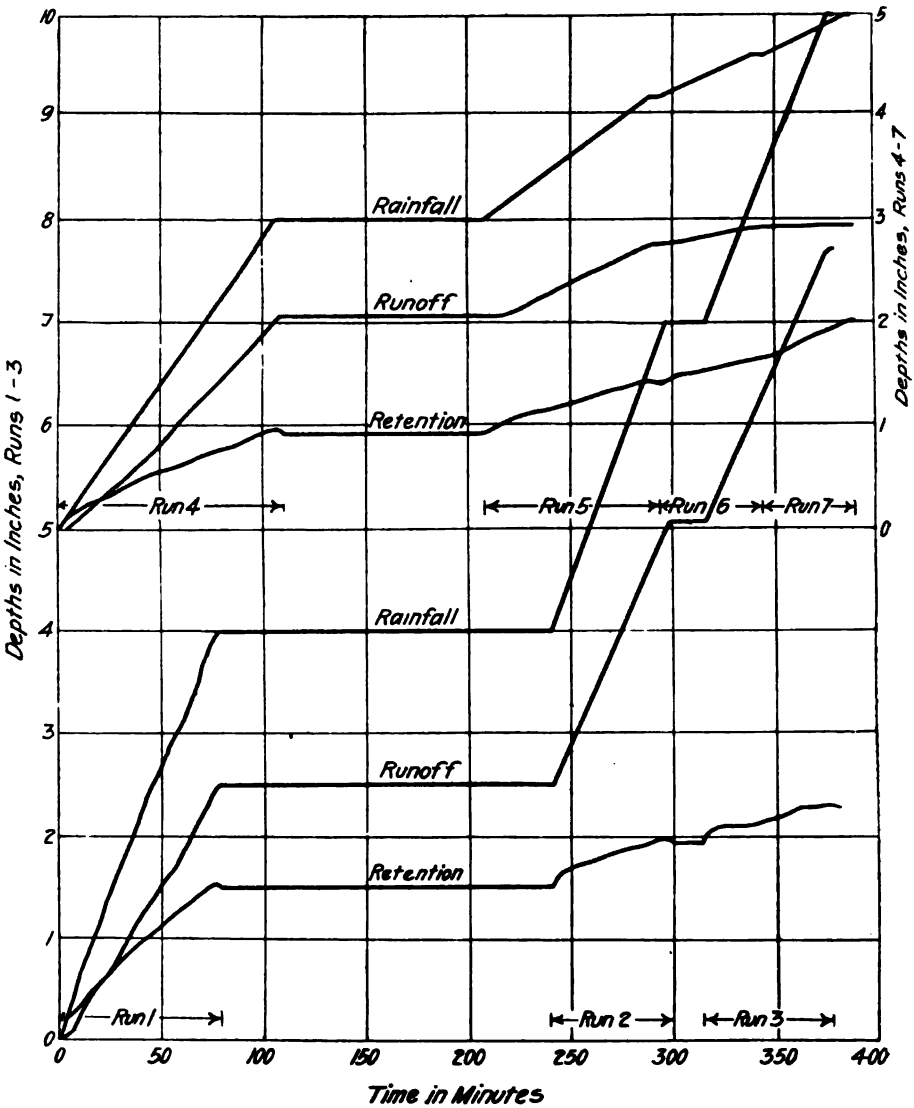


FIG. 16.—EXPERIMENT 5, MADE ON PLAT 1 AT THE TAYLORSVILLE DAM, AUGUST 9 TO 10, 1920.

The curves show the total rainfall, runoff, and retention up to any instant.

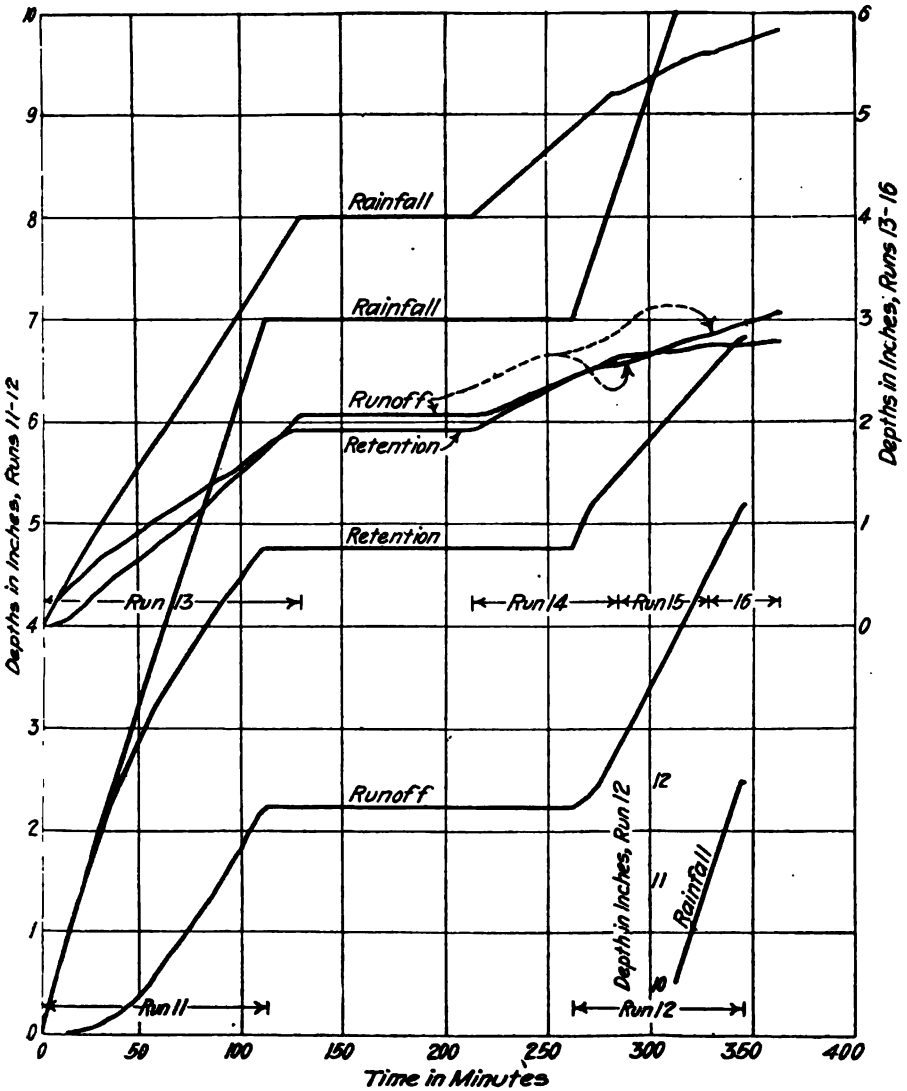


FIG. 17.—EXPERIMENT 6, MADE ON PLAT 2 AT THE TAYLORSVILLE DAM, AUGUST 11 TO 12, 1920.

The curves show the total rainfall, runoff, and retention up to any instant.

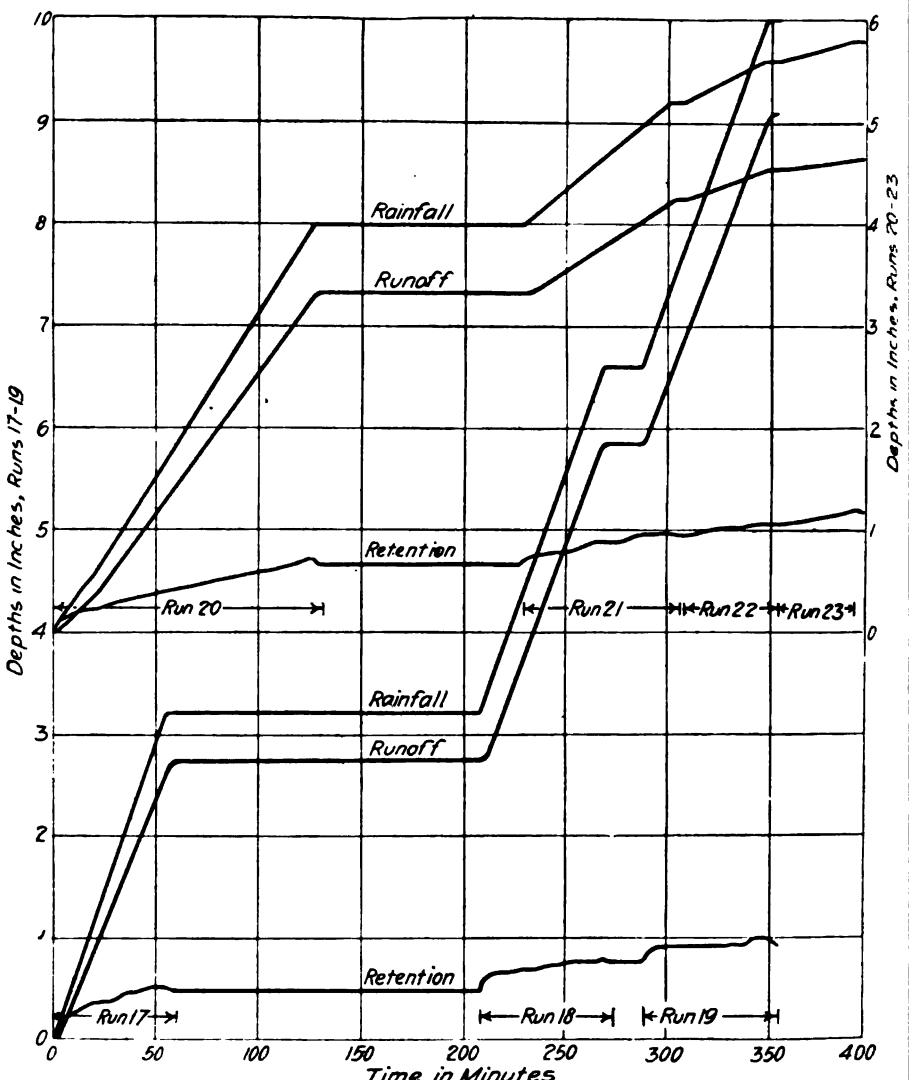


FIG. 18.—EXPERIMENT 7, MADE ON PLAT 3 AT THE TAYLORSVILLE DAM, AUGUST 16 TO 17, 1920.

The curves show the total rainfall, runoff, and retention up to any instant.

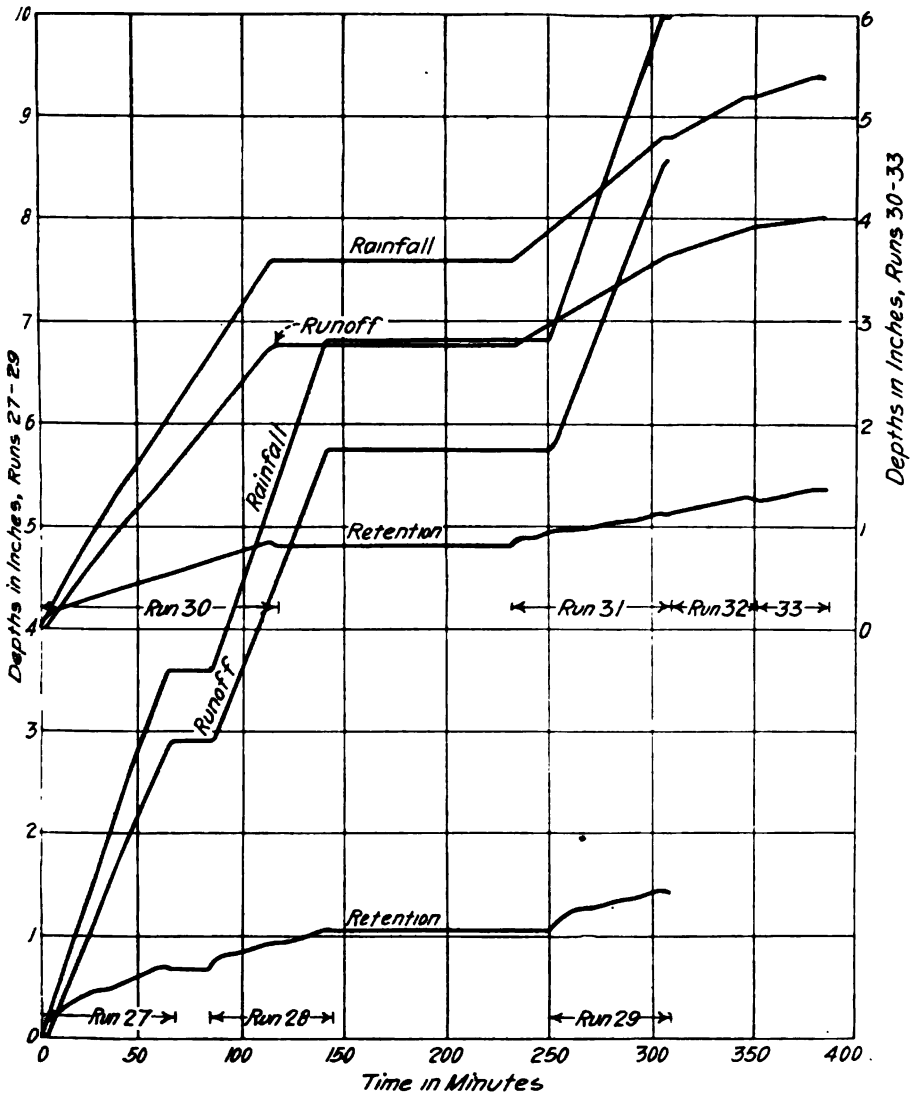


FIG. 19.—EXPERIMENT 8, MADE ON PLAT 4 AT THE TAYLORSVILLE DAM, AUGUST 18 TO 19, 1920.

The curves show the total rainfall, runoff, and retention up to any instant.

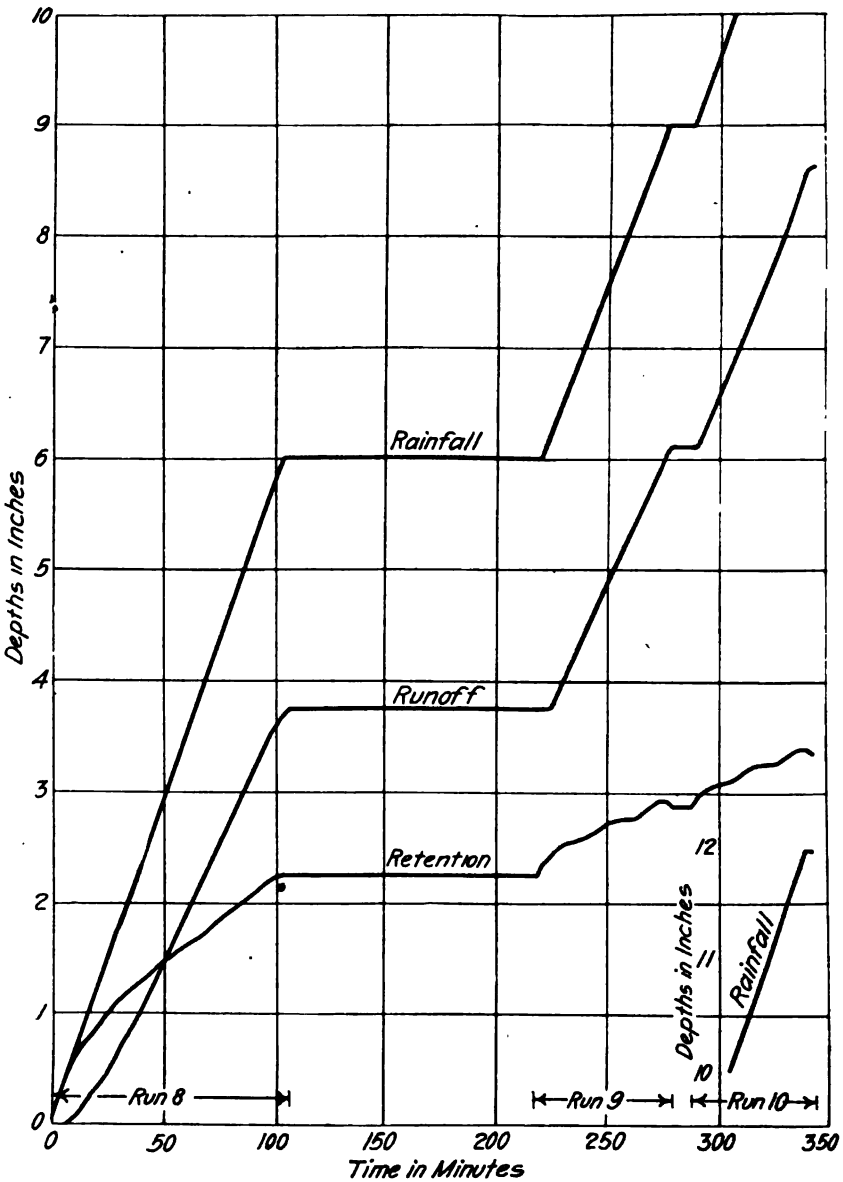


FIG. 20.—EXPERIMENT 9, MADE ON PLAT 1 AT THE TAYLORSVILLE DAM, OCTOBER 19, 1920.

The curves show the total rainfall, runoff, and retention up to any instant.

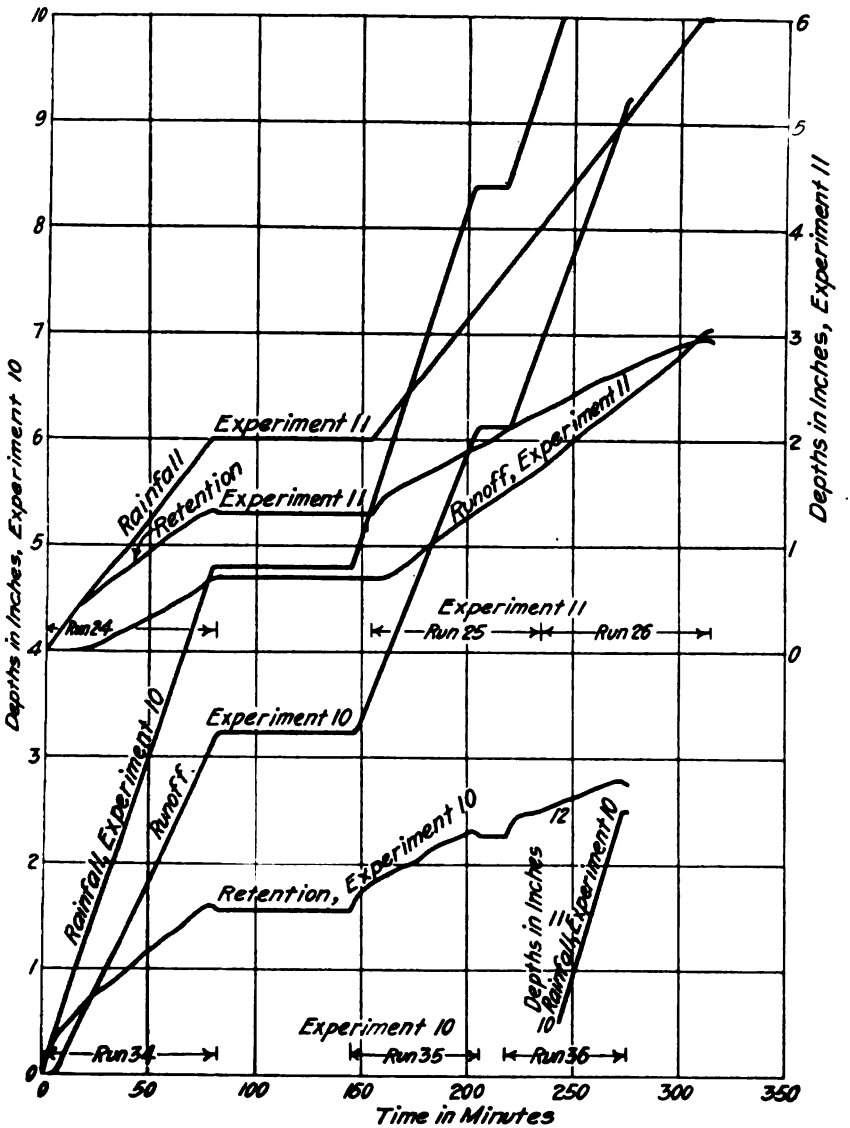


FIG. 21.—EXPERIMENTS 10 AND 11, MADE ON PLATS 4 AND 3 AT THE TAYLORSVILLE DAM, OCTOBER 21 AND 22, 1920.

The curves show the total rainfall, runoff, and retention up to any instant.

ments 9, 10, and 11 were made, than it was when the earlier experiments were begun. Consequently the data in table 18 is comparable with conditions occurring during intense summer thunderstorms, or cloudbursts, rather than to those existing during large winter storms.

It is interesting to note that although the total precipitation was unusually large in practically all of the experiments, the rates of retention when the experiments were finished were still comparatively large. This would indicate that the soil was capable of taking up considerable additional moisture. The lowest rate of retention given in table 18, 0.09 inches per hour on the level bare soil plat at Moraine Park, would correspond to a total absorption of 0.90 inches in 10 hours or 2.16 inches in 24 hours if continued for such periods of time. It should be noted that these rates of retention are lower, in most instances, than they would have been if the rates of precipitation had been higher, since it will be shown later that the rate of retention generally increases somewhat, if only slightly, as the rate of precipitation increases.

The values of retention include soil absorption, percolation, and evaporation. Since the experiments were run on comparatively warm summer days when some wind was blowing, it is possible that the evaporation may have amounted to as much as 0.05 of an inch an hour in certain instances, although it is not believed that it ever could have exceeded this appreciably. At Moraine Park, owing to the shallow surface soil, retention must have been mostly percolation after the first run of each experiment.

Moraine Park Sod

Experiment 2, made on the level sod plat at Moraine Park, shows an unusually large retention. Of the total precipitation, 11.66 inches in 1.09 hours, only 1.29 inches ran off, leaving a retention of 10.37 inches. By referring to table 16 it will be seen that a rate of precipitation of 4.17 inches per hour was applied for 30 minutes with no runoff whatever; that a rate of 14.4 inches per hour was then applied for 12.2 minutes before runoff began; that this rate was continued for about 13 minutes, obtaining an average rate of runoff of only 0.4 inches per hour; and that a rate of 20.4 inches per hour was then applied for about 10 minutes, causing a rate of runoff of only 7.4 inches per hour, or less than half of the rate of application. In the last run the

water was applied with a garden hose, the rate being determined by tank measurements immediately before and after the run. In the first two runs the sprinkling was somewhat intermittent, since only one sprinkling can was available. The times given above are the net sprinkling times, not including the intervals during which the can was being filled.

These extremely unusual results are due to the unusual soil conditions existing at this plat. As explained in chapter III the soil is loose and is filled with roots down to the depth where the material is mostly sand and gravel. Consequently the water can percolate downward at very great rates.

The data obtained by sprinkling agrees with the natural rainfall and runoff records given in the preceding chapter. It was there shown that runoff on this plat occurred very infrequently; and that when it did occur it was generally due to melting snow or to rain at times when the ground surface was frozen.

Moraine Park Bare Soil

Experiments 1, 3, and 4 were made on the bare soil plats at Moraine Park, numbers 1 and 3 being made on the level plat and number 4 on the sloping plat. The soil in the sloping plat when experiment 4 was begun was in about the same condition as the soil in the level plat when experiment 1 was started. It was comparatively dry and loose in both cases. When experiment 3 was started, however, the soil in plat 1, although dry, was hard and packed, and consequently in a much more impervious condition. This was caused by the trampling of a herd of cattle which was turned into the field in which the plats are located, immediately after experiment 1 was made, while the soil was still in a saturated condition. The sprinkling was intermittent during experiment 1 due to only one can being available, but was continuous during experiments 3 and 4.

By referring to table 18 it will be seen that the total retention during experiment 1 was considerably greater than the total during experiment 3, although the quantity of water applied in the latter instance was about 70 per cent greater than in the former. This, of course, would be expected due to the difference in soil conditions and in sprinkling methods. For experiment 4 the retention was 4.73 inches or only slightly more than for experiment 1, although the total precipitation was practically twice as great. The relatively smaller retention during ex-

periment 4 was due to the slope of the ground as well as to the difference in sprinkling methods.

The total runoff was 4.44 inches, or about 51 per cent of the total rainfall, during experiment 1; 12.30 inches, or about 83 per cent of the total rainfall, during experiment 3; and 12.07 inches, or about 72 per cent of the rainfall, during experiment 4.

Taylorsville Plats

Experiments 5 to 11, inclusive, were run on the plats at Taylorsville; numbers 5 and 9 on plat 1, number 6 on plat 2, numbers 7 and 11 on plat 3, and numbers 8 and 10 on plat 4. Plat 2 was spaded and raked before experiment 6 was begun, as previously noted. The other plats were in their natural condition. Sprinkling was continuous during all experiments.

Experiments 5 to 8, inclusive, were practically the same as regards time of sprinkling and total quantity of water applied, except that the total precipitation during number 6 was from 2 to 3 inches greater than during the others. Experiments 9 and 10 were similar, each being run in about the same time and at about the same rainfall intensity. Experiment 11 was run in about the same time as numbers 9 and 10 but the water was applied at a less intense rate.

Reference to experiments 5 to 8, inclusive, in table 18 shows that the retention on plat 2, in experiment 6, where the soil had been loosened, was more than twice as great as on plat 1, in experiment 5, where the soil was similar in composition and texture but had not been spaded; and about four times as great as on plats 3 and 4, experiments 7 and 8, where the soil was mostly clay and had not been loosened. The total retention on plat 2 was 9.88 inches or about 55 per cent of the rainfall; as against 4.15 inches or about 28 per cent of the rainfall, on plat 1, and about 2.45 inches or about 16 per cent of the rainfall, on plats 3 and 4. The total runoff amounted to 7.95 inches or about 45 per cent of the rainfall in experiment 6; to 10.65 inches or about 72 per cent of the rainfall in experiment 5; and to about 13.15 inches or about 84 per cent of the rainfall in experiments 7 and 8.

In experiment 9, made on plat 1 in October when the soil was slightly drier than in August, the retention amounted to 3.35 inches, or about 28 per cent of the precipitation. During runs 1 to 3 of experiment 5, which are comparable with experiment 9, the retention amounted to 2.30 inches or about 23 per cent of the rainfall. Experiments 10 and 11 also show greater values

of retention than were obtained in the parts of experiments 8 and 7 which are comparable. It is interesting to note that the retention during experiment 11 was slightly greater than during experiment 10 although the total precipitation in the latter was twice as great. This is due to the slightly greater sprinkling time during experiment 11. The rate of retention for plats 3 and 4 varies only slightly with the rate of rainfall, as will be shown later. Conditions during experiments 10 and 11 are shown in figure 21.

RAINFALL AND RUNOFF RATES ON SATURATED SOILS

The data in tables 16 and 17 is essentially data on rainfall, retention, and runoff on saturated soils. Although the soil was comparatively dry when the experiments were started in all cases, it soon became saturated to such a depth that runoff began. By the end of the first run the soil was generally saturated to such a depth that the rate of runoff caused by a given rate of precipitation was practically constant. This was true for all experiments except number 2, made on the sod plat at Moraine Park, where the soil conditions were unusual, as previously mentioned; and numbers 10 and 11, made on the hill plats at Taylorsville, in October, when the soil was somewhat drier than in the earlier experiments. At Taylorsville, the actual depths of saturation probably varied somewhat during the later runs of the other experiments, increasing as the work progressed. However, these variations may, for the present, be neglected. At Moraine Park the sand and gravel deposits, underlying the 2-foot layer of surface soil, afforded ready drainage and thus prevented the extension of saturated conditions below this depth.

Moraine Park Bare Soil

The rates of rainfall, retention, and runoff for the bare soil plats at Moraine Park, given in table 16, are shown graphically in figure 22, rates of rainfall being platted as ordinates against rates of runoff as abscissas. The rates of retention are represented by the horizontal or vertical distances from the platted points to the 45° line, curve D, drawn through the origin. This 45° line represents the limiting conditions of runoff. A point would fall on this line only when the runoff rate was equal to the rainfall rate. Points representing runs where the soil was not

saturated have been identified by placing near them the run numbers. Other points have not been numbered.

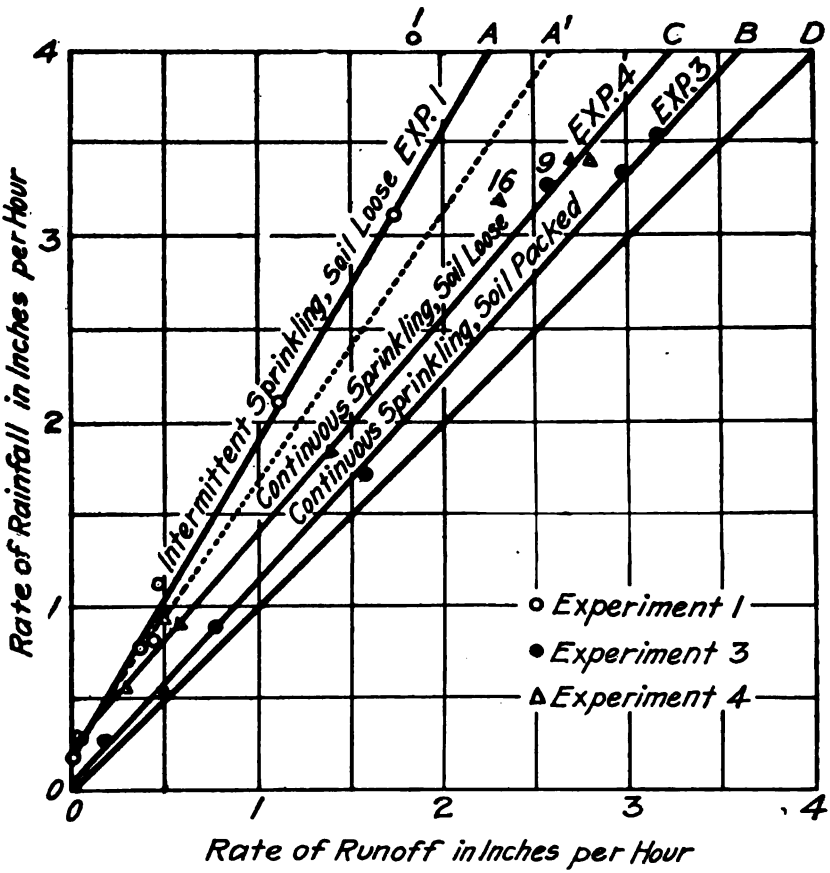


FIG. 22.—RATES OF RAINFALL AND RUNOFF ON BARE SOIL PLATS AT MORAINE PARK.

Runs in which the soil was not saturated are numbered. Experiments 1 and 3 were made on the level plat and experiment 4 on the sloping plat. Curve D is a 45 degree line.

For each experiment a line has been drawn so as to balance all points except those where the soil was not saturated. Consequently these lines show the variations in the rate of runoff caused by variations in the rate of rainfall, during the summer and fall, after the soil has become saturated by a fairly heavy and intense precipitation.

Variations in the rate of retention due to variations in the rate of rainfall are shown by the differences between the lines

representing the observations and the 45° line. Differences in the relation between rainfall and runoff for saturated soils due to variations in soil texture and in surface conditions are shown by the differences in the slope and in the location of curves A, B, and C.

Curve A averages the various runs of experiment 1, made on the level plat, curve B averages those of experiment 3, also made on the level plat, and curve C averages those of experiment 4, made on the sloping plat. It will be noticed that the rates of runoff corresponding to given rates of rainfall were considerably lower in experiment 1 than they were in experiment 3, due to the different method of sprinkling as well as to the different condition of the soil; also that they were considerably lower in experiment 1 than they were in experiment 4, due to the different method of sprinkling and to the different slopes of the surfaces.

In all three curves the rate of retention increases as the rate of rainfall increases. In curves B and C this is due entirely to the head caused by the greater depth of water on the ground, this increased head producing an appreciable effect because of the comparatively shallow depth of the surface soil. While the effect of varying head must have been fully as great in curve A as in curves B and C, the method of sprinkling and of calculating the rates of runoff also had some effect. If the sprinkling had been continuous in experiment 1, or if it had been possible to eliminate surface storage effects in calculating the rates of runoff, curve A would probably have fallen in some position intermediate between its present position and that of curve B, as at A′.

Probably the most interesting thing brought out by figure 22 is that the relation between rates of rainfall and runoff may be represented by straight lines; that is, that the relation represented by any one of these lines may be expressed by the straight line equation

$$y = sx + b$$

where y is the rate of rainfall, x is the rate of runoff, b is the intercept on the y axis, and s is the slope of the line.

It will be noticed that the value of b is about 0.20 inches per hour for curve A, 0.05 for curve B, and 0.24 for curve C. These are the rates of precipitation that can be maintained indefinitely on the Moraine Park bare soil plats, during the summer and fall when the soil is saturated, without any runoff whatever occur-

ing; that is, for these rates or lower rates, the water can percolate downward as fast as the rain falls.

Since the rate of percolation decreases with a decreasing temperature the value of *b* may be slightly smaller during the winter and spring. Allen Hazen,* speaking of friction losses in sand and gravel, says "I have found that the friction also varies with the temperature, being twice as great at the freezing point as at summer heat, both for coarse and fine sands." It is possible however, that in a shallow surface soil as at Moraine Park the loosening due to freezing and thawing may counteract to a certain extent the effect of the decrease in temperature.

Since curves A and C represent soil conditions comparable with those existing when the natural rainfall and runoff data was collected, the values of *b* determined from the sprinkling experiments furnish a satisfactory check on the conclusion reached in the preceding chapter; namely, that water can percolate through the surface soil on the bare soil plats, when the ground is saturated, at a rate as great as 0.25 of an inch an hour.

Taylorsville Plats

Figure 23 shows the Taylorsville data, contained in table 17, platted in the same manner as in figure 22. Curve D is the 45° line as before. Curve E averages the data taken on plat 1, curve F averages that taken on plat 2, and curve G averages that taken on plats 3 and 4. The conditions on plats 3 and 4 are practically identical, so that it is not necessary to draw a line for each plat. In platting the points, however, different symbols were used for the two plats, so that the agreement of the data may be seen. Sprinkling was continuous during all runs.

The increased slope of curve F over that of curve E shows the increased retention obtained by spading, or loosening, the soil. It will be noticed that the rate of retention increases considerably as the rate of precipitation increases, in the case of plat 2; but that it is practically constant in the case of plat 1. The slightly higher rates of runoff shown by curve G over those shown by curve E are due to the slightly greater impermeability of the clay soil at the hill plats. The rate of retention at the hill plats is similar to that at plat 1, in that it increases only

*Some Physical Properties of Sands and Gravels, by Allen Hazen, Massachusetts State Board of Health, Boston, Massachusetts, Twenty-fourth Annual Report, page 553, 1892.

slightly with the increasing rate of precipitation. It is comparatively small throughout the range covered by the data.

Here, also, the relations between rates of rainfall and runoff for saturated soil may be shown by straight lines. The value of *b* is seen to be 0.30 inches per hour for curves E and F and about 0.20 inches per hour for curve G, meaning that precipitation can

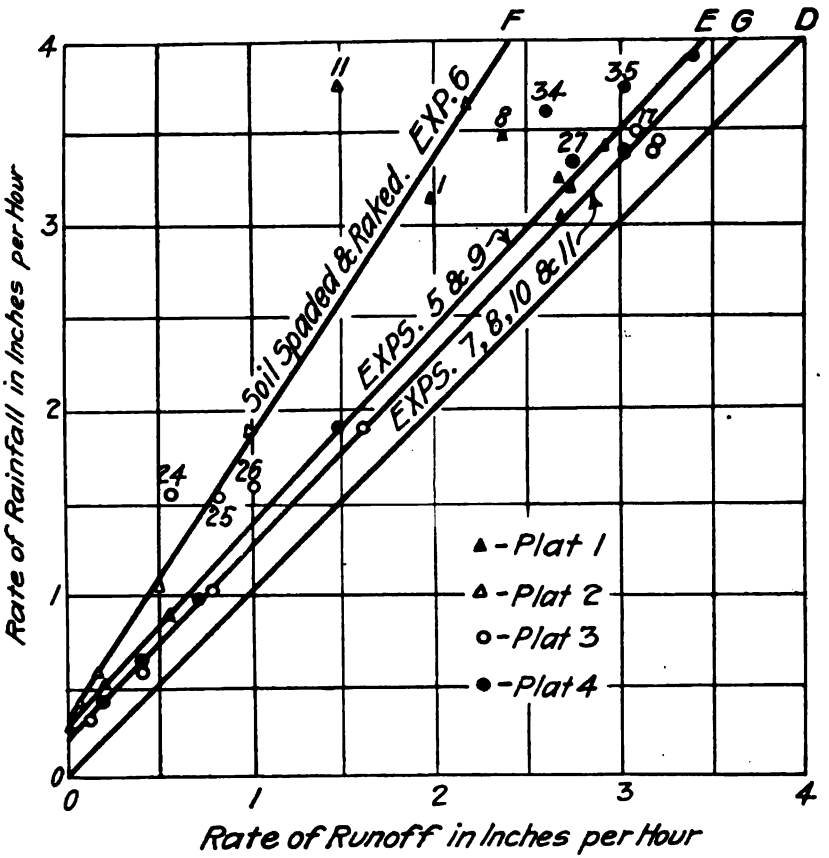


FIG. 23.—RATES OF RAINFALL AND RUNOFF AT THE TAYLORSVILLE DAM.

Runs in which the soil was not saturated are numbered. Note the increased retention obtained by spading the soil. Curve D is a 45 degree line.

occur at these rates, during the summer and fall when the surface soil is saturated, without any runoff taking place. It should be noted that these values are practically the same as those obtained at Moraine Park notwithstanding the difference in subsoil. For winter and spring conditions, however, the values of

b at Taylorsville may be lower than at Moraine Park, due to the greater depth of saturation and the lack of adequate soil drainage.

Average Relations

In figure 24 the various curves of figures 22 and 23 have been brought together. Points have not been shown since they would

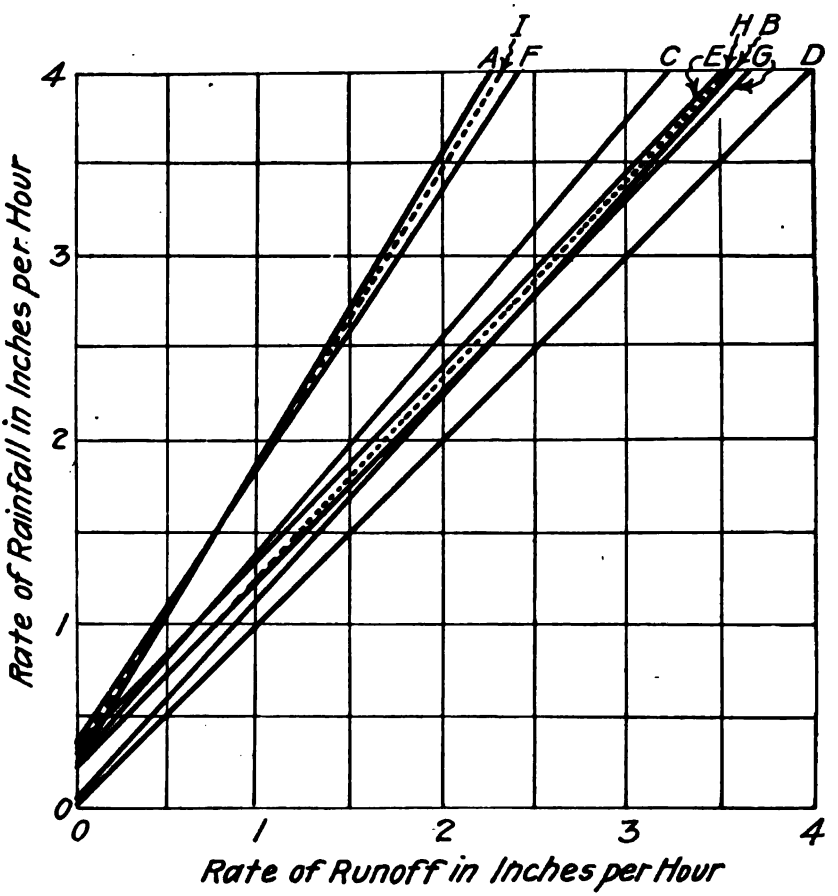


FIG. 24.—AVERAGE RELATIONS BETWEEN RATES OF RAINFALL AND RUNOFF, SOIL SATURATED.

The two dotted lines represent the average relations shown by the two groups of full lines. For identification of full lines see figures 22 and 23.

only confuse the diagrams. It will be noticed that the lines fall into two separate groups; first, curves A and F, and, second, curves B, C, E, and G. Curve H has been added to represent the

average relation between rainfall and runoff rates shown by the lines of the second group, and curve I has similarly been drawn to represent the average relation shown by those of the first group.

The relation shown by curve H may be expressed by the formula

$$y = 1.07x + 0.20 \qquad (1)$$

where y is the rate of rainfall and x the rate of runoff, both expressed in inches per hour. Since y is the cause and x the effect it seems more advantageous to change equation 1 to the form

$$x = 0.93\ (y - 0.20) \qquad (2)$$

It will be noticed that the coefficient of the quantity $(y - 0.20)$ is not greatly different from unity; or in other words that curve H is nearly parallel to curve D. This means that the rate of retention increases only slowly with an increasing rate of precipitation. For practical purposes this variation in the rate of retention may be neglected. Taking the value of the retention as 0.30 inches per hour, a value corresponding to a rate of precipitation of 1.50 inches per hour by curve H, the relation between rates of rainfall and runoff may be expressed by the equation

$$x = y - 0.30 \qquad (3)$$

The line representing equation 3 would be parallel to curve D and a constant distance above, equivalent to 0.30 inches per hour.

Consequently, during the summer and fall when the ground is saturated, rates of runoff from soils similar to those represented by curves B, C, E, and G may be estimated by simply deducting 0.30 inches per hour from the rates of rainfall.

Curve I, representing the average relation for curves A and F, may be expressed by the equation

$$y = 1.60x + 0.25 \qquad (4)$$

or, solving for x,

$$x = 0.62\ (y - 0.25) \qquad (5)$$

The retention would then be expressed by the equation

$$y - x = 0.60x + 0.25 \qquad (6)$$

This time the rate of retention increases appreciably as the rate of rainfall increases; and, consequently, it will not be advisable to replace the slope coefficient 1.60 by unity.

Equations 3, 5, and 6 are applicable during the summer and fall, on areas similar to those where the experiments were made, after the surface soil has become saturated by a precipitation of three or four inches falling in one of two days. The data

given in the preceding chapter indicates that equation 3 is applicable on the sloping bare soil at Moraine Park during the winter and spring after a half an inch or an inch has fallen, although it probably will give rates of runoff slightly too low at such times. The rates of percolation during the winter and spring on plats 1, 3, and 4 at Taylorsville, after an inch has fallen, may differ somewhat from the 0.30 of an inch an hour shown by equation 3, no data being available for these plats for such seasons of the year. The rate of percolation on plat 2 probably decreases as the soil becomes packed by the winter and spring rains.

The average relation between rates of rainfall and runoff for the Miami Valley during the summer and fall when the surface soil is saturated, probably lies somewhere between those shown by equations 3 and 5. It is, of course, very difficult to estimate the average soil conditions over a large drainage area. However, considering that practically throughout the valley the amount of vegetation exceeds that on plats 1, 3, and 4 at Taylorsville, and also that the greater part of the land is cultivated, it seems fairly certain that the average rate of retention would exceed that used in determining equation 3. It does not seem possible though, that the retention could amount to as much as that obtained on plat 2 at Taylorsville, where the soil had been thoroughly spaded and raked just before the experiment was made.

RAINFALL AND RUNOFF RATES, SOIL NOT SATURATED

When the ground is not saturated the relation between rates of rainfall and runoff varies greatly with the amount of moisture in the soil, as well as with the soil texture and the surface conditions. On a given plat the drier the soil the greater will be the rate of retention and the smaller will be the rate of runoff corresponding to a given rate of rainfall.

Experiments 10 and 11, made on plats 4 and 3 at Taylorsville, furnish data on the relation between rates of rainfall and runoff for different amounts of soil moisture. Since the soil and surface conditions are the same at these two plats, and since the amount of moisture present when experiments 10 and 11 were started was the same, it is possible to select portions of these experiments in which the amount of soil moisture present was the same for both plats. The rates of rainfall and runoff can then be calculated for these portions for both experiments and

the difference in the runoff rate for a given portion will be due entirely to the difference in the rainfall rate. The rates may then be platted as in figures 22 and 23 and curves may be drawn to represent the relations for the different amounts of soil moisture.

In figure 25 the circles show the rainfall and runoff rates determined in this manner for the following ranges of retention.

From 0.00 to 0.65 inches
From 0.65 to 1.30 inches
From 1.65 to 2.30 inches
From 2.40 to 2.75 inches

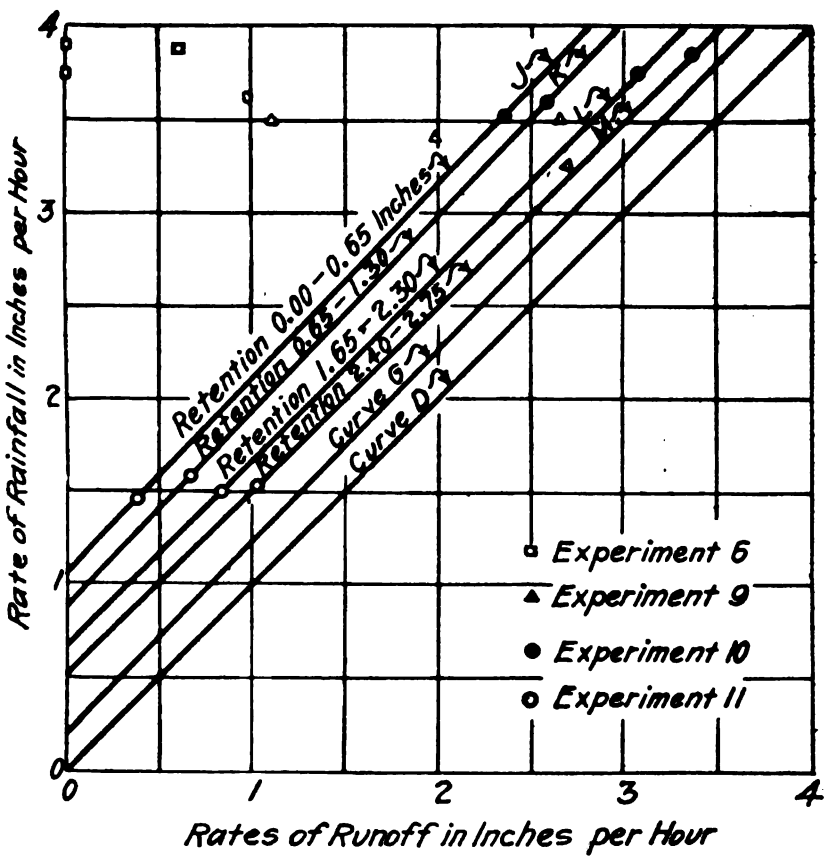


FIG. 25.—RATES OF RAINFALL AND RUNOFF AT THE TAYLORSVILLE DAM WHILE SOIL IS BECOMING SATURATED.

Lines have been drawn through points calculated from experiments 10 and 11.

The upper set of circles were determined from experiment 10 and the lower set, from experiment 11. Straight lines have been drawn through the points for each range in retention, curves J, K, L, and M; and a 45° line, curve D, has been added as in figures 22 to 24. For comparative purposes, curve G of figure 23, representing saturated surface soil conditions in plats 3 and 4, has also been added. Since only two points are available for each line, it is, of course, not known whether they should be straight or curved. They were drawn straight because it has been shown that for saturated soil conditions the curves are straight.

It will be noticed that curves J, K, L, and M are spaced rather uniformly, and that they are nearly parallel to curve D, the 45° line. However, if the experiments had been carried further, it is likely that the succeeding lines would have been increasingly closer together as they approached curve G, since curve G was based primarily on experiments in which the total sprinkling time, as well as the amount of moisture in the soil when the observations were started, was greater than in experiments 10 and 11.

Curves J, K, L, and M illustrate, for plats 3 and 4, the variations in runoff and retention rates caused by variations in rainfall rates and in total retention. Retention rates are shown by the horizontal or vertical distances from the various lines to curve D. The variations in runoff and retention rates due to variations in soil texture may be indicated by showing, on figure 25, points calculated for plat 1 for similar ranges in retention during experiment 9. The soil in plat 1 when experiment 9 was begun contained about the same quantity of moisture as the soil in plats 3 and 4 when experiments 10 and 11 were started. Points calculated for experiment 9 are shown by the triangles in figure 25.

It will be noticed that the triangle corresponding to a range in retention from 0.00 to 0.65 inches falls a considerable distance to the left of curve J, thus indicating a considerably greater rate of retention and a correspondingly smaller rate of runoff for plat 1. The succeeding points, however, fall increasingly closer to the curves for plats 3 and 4. The last point, corresponding to a retention from 2.40 to 2.75 inches, is very close to curve M. This means that at Taylorsville the runoff from the clay soils on the hills is appreciably greater than the runoff from the loam

in the valley, when the soil is dry, but not materially different when the soil is saturated.

The effect of cultivation on retention and runoff may be shown on figure 25 by platting points computed from experiment 6, made on plat 2 at Taylorsville. As previously noted the soil in this plat was spaded and raked before the experiment was started. Points computed for the ranges in retention used in determining curves J, K, L, and M, are shown by the squares in the upper part of the diagram. It will be noticed that the points corresponding to ranges from 0.00 to 0.65 inches and from 0.65 to 1.30 inches, show no runoff whatever. Although the curves in figure 17 show the runoff as beginning when the retention had amounted to about an inch, the average rate before the retention reached 1.30 inches was so small that it could not be shown in figure 25. The points corresponding to ranges in retention from 1.65 to 2.30 and from 2.40 to 2.75 inches both fall to the left of the point calculated from experiment 9, plat 1, for the range from 0.00 to 0.65 inches. This illustrates the relatively great amount of retention obtained by cultivation. Reference to figure 17 shows that after runoff did begin on plat 2, the rate increased gradually throughout the first run. That the increase was gradual rather than abrupt was due to the presence of air in the soil.

The ranges in retention for which the points were computed are noted on the curves in figure 25. Points corresponding to similar ranges during other experiments, or during actual rainfalls, will fall on these lines only when the soil conditions, as regards texture, temperature, and moisture, at the beginning of the precipitation are the same as they were in experiments 10 and 11. Consequently, in order to use curves J, K, L, and M in calculating runoff from rainfall, it will be necessary to estimate the condition of the soil when the rainfall begins. This estimate can probably be made closely enough that the runoff rate will be determined with a fair degree of accuracy. For instance, if it is estimated that when a rainfall of 2 inches per hour, lasting an hour, began, the condition of the soil was the same as in experiments 10 and 11, the runoff rate of about an inch an hour, shown by the curves in figure 25, is probably accurate within 25 per cent or within a quarter of an inch an hour. This uncertainty would decrease in amount as the soil became saturated. While an uncertainty as great as 25 per cent is un-

desirable, it is doubtful if an estimate based on judgment alone would be as accurate as one based on the curves of figure 25.

The data secured at Moraine Park is hardly sufficient to warrant the preparation of a diagram such as figure 25. However, the differences in retention and runoff due to variations in soil texture and surface slope may be studied in figures 13, 14, and 15, illustrating experiments 1, 3, and 4. Experiment 1 was made on the level bare soil plat in June, experiment 3 on the same plat in July, after the soil had been trampled and packed by cattle, and experiment 4 on the sloping bare soil plat when the soil was comparatively loose. Considering the first run of each experiment the retention rates are seen to have been considerably greater, and the runoff rates considerably less, during experiment 1 than during experiments 3 and 4. During experiment 4 the retention rates were slightly greater, and the runoff rates slightly less, than during experiment 3. This means that the increase in runoff due to the trampling and packing of the soil was slightly greater than that due to the increase in the slope of the surface.

The average rate of retention of 0.90 inches per hour obtained on the sloping plat in run 16, for a period of an hour and 15 minutes, checks the conclusion reached in the preceding chapter that water can be absorbed by the bare soil at times during the summer when the soil is unusually dry, at a rate as great as 1.00 inch per hour for intervals as long as 30 minutes.

CONDITIONS BEFORE RUNOFF BEGINS

A knowledge of the conditions necessary before runoff begins is valuable in studying rainfall and runoff. During many showers of comparatively short duration no runoff takes place although the intensity of the precipitation may be relatively great. During other showers of longer duration and lesser intensity similar conditions exist as regards runoff. In order for runoff to begin it will be necessary for two conditions to be fulfilled. First, the precipitation must occur at a rate greater than the rate at which it can be absorbed by the soil; and, second, the excess rate must continue long enough to fill the surface storage available by reason of the small depressions in the surface, accumulations of dead grass or leaves, growing vegetation, and other factors. The relative importance of these two conditions, of course, varies with the soil and surface characteristics. If the soil is bare and free from depressions, rates of precipitation and

soil absorption are predominant. If the soil is covered with a heavy sod or a deep deposit of forest litter, surface storage is the determining factor.

While it is not possible to differentiate between these two factors in a given instance, it is interesting to discuss their combined effect. Referring to run 1 of table 16 made on the level bare soil at Moraine Park when the ground was dry, it is seen that a rate of rainfall of 4.25 inches per hour caused runoff to begin in 2.0 minutes, or after a total of 0.14 inches had fallen. Run 1 of table 19, made on similar soil on the same day, showed that a rate of rainfall of 3.65 inches per hour caused runoff to begin in 2.5 minutes, or after a total of 0.15 inches had fallen. Run 16 of table 16, made on the sloping bare soil plat when the ground was dry, showed that a rate of 3.00 inches per hour resulted in runoff after 3.5 minutes, or after the total amounted to .18 inches. These results confirm the conclusion reached in the preceding chapter; namely, that water cannot be absorbed by the bare soil at Moraine Park at any time, no matter how dry it is, at a rate as great as 3.00 inches per hour for periods as long as 5 minutes. The apparent exception to this, indicated by run 9 of table 16, in which a rate of 3.65 inches did not cause runoff until 5.5 minutes, is due to the different condition of the surface, the surface in this instance containing a considerably greater number of small depressions.

Runs 1, 17, and 27 of table 17, made in August when the ground was about as dry as in the runs mentioned above, show that the soil at Taylorsville in plats 1, 3, and 4, is about the same as at Moraine Park as regards beginning of runoff. However, run 11, made on plat 2 where the soil had been spaded, shows a great difference. In this case a rate of 3.90 inches per hour did not cause a measurable quantity of runoff for 22 minutes, or until the total precipitation had amounted to 1.43 inches. The following morning, when the soil was practically saturated a rate of 1.85 inches per hour resulted in runoff in 6 minutes, or after 0.18 inches had fallen.

Runs 8, 24, and 34 of table 17, made on plats 1, 3, and 4 in October, when the soil was somewhat drier than in August, show slightly greater values of retention preceding runoff, than do runs 1, 17, and 27. Other runs of tables 16 and 17 show the conditions before runoff when the ground is practically saturated.

Figure 26 shows graphically the data discussed above. Times

in minutes required for runoff to begin are platted as abscissas against the corresponding rates of precipitation as ordinates. Points corresponding to all runs in table 19 have been platted, but only those corresponding to the first run of each day have been platted from tables 16 and 17, since runs made on the same day were frequently only a few minutes apart. Different sym-

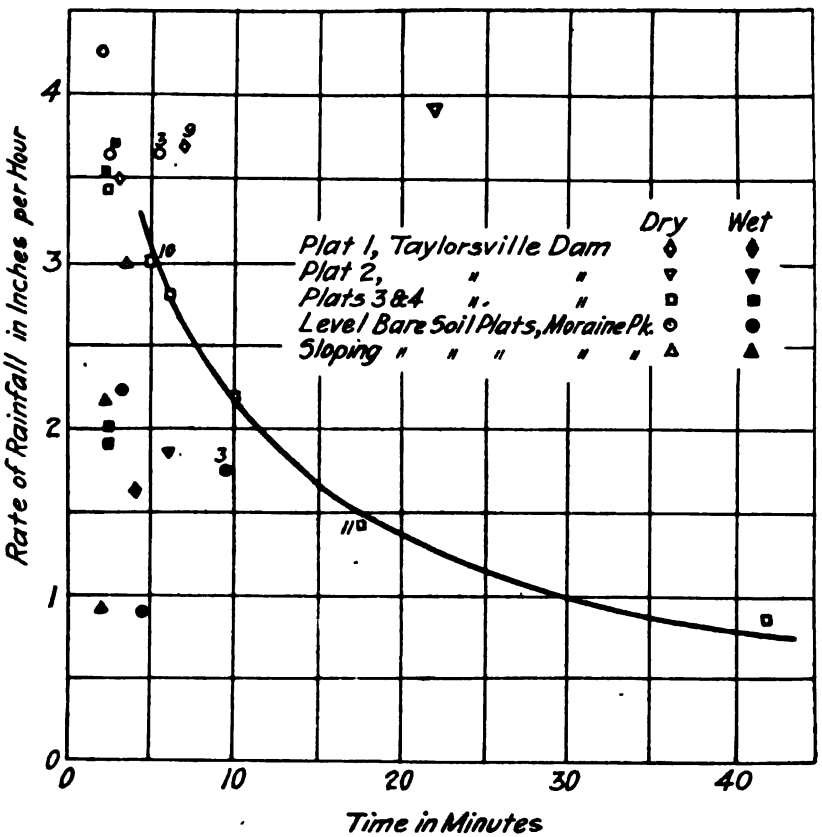


FIG. 26.—INTENSITY AND DURATION OF RAINFALL BEFORE RUNOFF BEGINS.

Data secured at the Moraine Park and Taylorsville plats. The curve represents conditions at plats 3 and 4 at Taylorsville when the soil is dry.

bols have been used to indicate the various plats on which the data was secured. Where the ground was wet when the rainfall began the points have been blackened; where it was dry, they have been left white. In one or two instances the experiment number has been placed near the point in order to indicate a different soil condition.

Points representing runs 24 and 34 of table 17, and 2, 3, and 4 of table 19, made at the Taylorsville hill plats in October, have been balanced by a line, since for these runs the soil and surface conditions were practically the same. Points secured at Moraine Park do not cover a sufficient range to determine a curve.

The amount of the surface storage in a given instance is indicated by the height of the hump, or peak, at the end of the retention curve, see figures 13 to 21, inclusive, caused by the draining off of the water after the precipitation ceased. The amounts are small in all cases, as would be expected since there was relatively little vegetation on any of the plats for which mass curves were platted. The quantities vary from practically nothing to about 0.07 of an inch.

CHAPTER V.—MONTHLY, SEASONAL, AND ANNUAL RAINFALL AND RUNOFF

INTRODUCTORY

General information regarding monthly, seasonal, and annual rainfall and runoff, their distribution throughout the year, their extreme variations, and the normal, monthly, seasonal, and annual amounts, are of importance in most hydraulic engineering work.

Criticisms are often made of the method of discussing rainfall and runoff by monthly, seasonal, or annual periods. These, as a rule, are based on the condition that the division date between periods may fall within a time of storm rainfall, or of flood runoff; or that due to snow accumulations, or ground water storage, precipitation during one period may affect the runoff in the following period. These objections, of course, are of more importance as regards studies based on the shorter periods. They also are of more importance with respect to studies of the larger drainage areas, inasmuch as flood runoff on the smaller areas is more nearly coincident with storm rainfall.

Such criticisms are well founded and should be borne in mind. However, they apply principally to theoretical studies of laws governing runoff rather than to particular engineering problems. Because such studies do not lead to the discovery of the laws of runoff is no reason why they should be wholly discontinued.

In making studies of seasonal and annual rainfall and runoff the above objections may be partially met by using the "water year" rather than the calendar year, and by a judicious division of the year into seasons, or periods. Rafter, in his studies of rainfall and runoff,* used the water year ending November 30. He divided the year into three periods, namely, the storage period, including the months from December to May, the growing period, including the months from June to August; and the replenishing period, including the months from September to No-

*The Relation of Rainfall to Runoff, by George W. Rafter, U. S. G. S. Water Supply Paper 80, 1903; also Hydrology of the State of New York. by George W. Rafter, Bulletin 85, New York State Museum.

vember. For conditions in the Miami Valley, the year ending September 30, which has been adopted by the Water Resources Branch of the U. S. Geological Survey, seems to be more satisfactory than the year ending November 30. It also seems better to consider the months from October to December as the replenishing period, the months from January to April as the storage period, and the months from May to September as the growing period.

This chapter will take up the studies of monthly, seasonal, and annual rainfall, runoff, percolation, and evaporation, which have been made for some of the drainage areas in the Miami Valley. A method of showing hydrological conditions by means of mass curves will also be described.

If the amount of water stored in the ground, or on the ground is the same at the beginning and ending of a period of time, the difference between the total rainfall and the total runoff during this period must be equal to the total evaporation, using the term evaporation to include plant transpiration and evaporation of precipitation intercepted by vegetation as well as direct evaporation from soil or water surfaces. Studies of ground water flow indicate that in the Miami Valley variations in the amount of

Table 20.—Stations Used in Studies of Rainfall and Runoff

Station	Stream	Drainage Area	Records Available*
		Square Miles	Years, Inclusive
Sidney	Miami River	555	1915–1919
Lockington	Loramie Creek	255	1916–1919
Piqua	Miami River	842	1912–1919
Tadmor	Miami River	1128	1915–1919
Pleasant Hill	Stillwater River	453	1917–1919
West Milton	Stillwater River	600	1915–1919
Springfield	Buck Creek	163	1915–1919
Springfield	Mad River	488	1915–1919
Wright	Mad River	652	1915–1919
Dayton	Miami River	2525	1894–1919
Franklin	Miami River	2785	1917–1919
Germantown	Twin Creek	272	1915–1919
Seven Mile	Seven Mile Creek	128	1915–1919
Hamilton	Four Mile Creek	178	1915–1919
Hamilton	Miami River	3672	1911–1919

*Years ending September 30.

water in the ground at the end of the water year are relatively small proportions of the yearly evaporation. Consequently, in the following tables and discussions dealing with annual quantities the term evaporation is used to mean the difference between the rainfall and runoff. However, in the studies of seasonal and

monthly values the term retention has been used, since in these cases variations in the amount of water in the ground are comparatively large.

COMPILATION OF THE DATA

Table 20 gives the gaging stations for which the data was compiled, the streams on which they are located, the areas drained, and the period of years for which records are available, the division into years being made on September 30 instead of on December 31. It will be noted that records for full years prior to 1915 are available only for the Piqua, Dayton, and Hamilton stations.

The annual rainfall, runoff, retention, and ratio of runoff to rainfall, were tabulated for all records available, for each station in table 20, except Piqua. On account of unreliable gage height data at Piqua for some of the earlier years and for a part of the year 1918, the quantities were tabulated for the years 1915, 1916, 1917, and 1919 only. The annual, seasonal, and monthly rainfall, runoff, retention, ratio of runoff to rainfall, and temperature had been compiled for the entire record at the Dayton station just before the 1919 data was compiled. Since the 1919 values do not differ materially from the averages based on the 25-year record the studies have not been revised so as to include the 1919 data, except in the case of table 21.

The proportions of annual runoff which appear as surface or flood runoff and as low water or ground water flow, were determined for the Dayton, Wright, West Milton, and Buck Creek stations. The Dayton station was chosen because of its comparatively long record and because it is representative of the average conditions throughout the Miami Valley. The other stations were chosen because a cursory examination of the records, as well as the study of flood runoff given in the following chapters, indicated that the surface runoff from their drainage areas varies considerably. Mass curves were drawn only for the drainage area of Mad River above Wright.

The annual, seasonal, and monthly rainfall given in the tabulations are averages over the drainage areas above the stations, not the amounts recorded at the stations themselves; and are for the years ending September 30, rather than for the calendar years. For the years 1915 to 1919, inclusive, the annual amounts were determined by planimeter measurements on maps showing lines of equal annual rainfall. The annual, seasonal, and monthly

Table 21.—Annual Rainfall, Runoff, and Evaporation in the Miami Valley

Stations	Rainfall Inches	Runoff Inches	Ratio of Runoff to Rainfall per cent	Evaporation Inches	Rainfall Inches	Runoff Inches	Ratio of Runoff to Rainfall per cent	Evaporation Inches	Rainfall Inches	Runoff Inches	Ratio of Runoff to Rainfall per cent	Evaporation Inches
	1915				1916				1917			
Sidney	40.4	12.2	30.2	28.2	38.3	19.4	50.6	18.9	35.2	8.98	25.5	26.2
Lockington	41.2	.....	.....	.....	37.9	20.8	54.9	17.1	35.0	13.8	39.4	21.2
Piqua	41.0	14.3	34.9	26.7	38.4	19.6	51.9	18.8	35.2	10.4	29.6	24.8
Tadmor	41.7	13.5	32.4	28.2	39.1	19.7	50.4	19.4	35.3	11.9	33.7	23.4
Pleasant Hill	44.8	.....	.....	.....	37.3	.....	.....	.....	38.3	13.6	35.5	24.7
West Milton	43.7	12.6	28.8	31.1	37.6	17.6	46.8	20.0	37.1	12.3	33.2	24.8
Buck Creek	40.1	8.3	20.7	31.8	42.2	14.8	35.1	27.4	34.2	10.2	29.8	24.0
Springfield	41.2	11.5	27.9	29.7	43.8	17.4	39.7	26.4	36.6	12.0	32.8	24.6
Wright	40.8	12.0	29.4	28.8	42.8	19.4	45.3	23.4	36.4	13.5	37.1	22.9
Dayton	41.8	12.1	28.9	29.7	39.9	19.2	48.1	20.7	35.7	11.4	31.9	24.3
Franklin	41.8	.....	.....	.....	39.8	.....	.....	.....	35.8	11.9	33.2	23.9
Germantown	41.6	15.8	38.0	25.8	38.6	18.6	48.2	20.0	37.4	13.7	36.6	23.7
Seven Mile	41.6	17.5	42.1	24.1	39.1	23.5	60.1	15.6	39.0	17.0	43.6	22.0
Four Mile	41.0	16.3	39.7	24.7	39.6	24.8	62.6	14.8	38.0	13.8	36.3	24.2
Hamilton	41.7	11.2	26.9	30.5	39.8	18.8	47.2	21.0	36.4	11.6	31.9	24.8
	1918				1919				Average			
Sidney	35.1	9.1	25.9	26.0	32.8	9.0	27.4	23.8	36.3	11.7	32.2	24.6
Lockington	33.7	8.7	25.8	25.0	30.9	.....	.....	.....	35.8	13.3	37.2	22.5
Piqua	34.8	.....	.....	.....	32.3	9.4	29.1	22.9	36.4	12.6	34.6	23.8
Tadmor	36.0	9.2	25.5	26.8	33.1	11.5	34.7	21.6	37.0	13.2	35.7	23.8
Pleasant Hill	35.2	10.1	28.7	25.1	33.4	12.5	37.4	20.9	37.8	13.3	35.2	24.5
West Milton	35.4	13.3	37.6	22.1	33.5	12.0	34.0	22.1	37.5	13.6	36.3	23.9
Buck Creek	38.8	10.1	26.0	28.7	37.6	11.0	29.3	26.6	38.6	10.8	28.0	27.8
Springfield	38.6	10.5	27.2	28.1	38.5	12.8	33.2	25.7	39.7	12.8	32.2	26.9
Wright	39.2	11.1	28.3	28.1	38.1	12.9	33.9	25.2	39.5	13.8	34.9	25.7
Dayton	36.8	9.4	25.6	27.4	34.6	11.2	32.4	23.4	37.8	12.6	33.3	25.2
Franklin	36.9	8.9	24.1	28.0	34.6	11.2	32.4	23.4	37.8	12.6	33.3	25.2
Germantown	39.5	12.1	30.7	27.4	34.5	12.9	37.4	21.6	38.3	14.7	38.4	23.6
Seven Mile	43.4	21.9	50.5	21.5	34.1	17.1	50.1	17.0	39.5	19.4	49.1	20.1
Four Mile	41.6	10.6	25.5	31.0	35.4	10.7	30.2	24.7	39.2	15.3	39.0	23.9
Hamilton	37.9	9.6	25.3	28.3	34.5	11.5	33.3	23.0	38.0	12.5	32.9	25.5

rainfall for years prior to 1915, and the seasonal and monthly values for the years 1915 to 1918, where it was only necessary to obtain data for the Dayton and Hamilton stations, were obtained by averaging directly the records of all stations on the given drainage areas. While the latter method does not consider the distribution of stations, comparisons of the results obtained, with those obtained by the planimeter measurements, showed that for such large areas and with so many stations, the results by the shorter method are not appreciably in error.

The values of annual, seasonal, and monthly runoff were calculated from the daily stream flow records, except in the case of the 1911 and 1912 records at Hamilton. These were obtained from the U. S. Geological Survey water supply papers, proper corrections being made for the flow in the Miami and Erie Canal, which is not included in the government data.

In the studies of the relation of temperature to runoff the records at the Dayton co-operative station were utilized. It was not considered necessary to calculate the average temperature over the drainage area, inasmuch as any difference which may exist tends to be constant in amount, algebraically as well as arithmetically, and also tends to be relatively small.

The method of estimating the proportions of annual runoff which appear as surface or flood flow and as low water or ground water flow, and the method of drawing mass curves, will be described later.

ANNUAL RAINFALL AND RUNOFF

Records for Years 1915-1919

Table 21 gives the annual rainfall, runoff, ratio of runoff to rainfall, and evaporation for all stations at which stream flow records are being compiled, for the years 1915 to 1919, inclusive. The average values, although very uncertain due to the shortness of the period, are also included. Studies based on the 25-year record at the Dayton station, discussed later, show that no one of these years was greatly different from normal. In order that the average values should be comparable throughout, missing records at the Lockington, Piqua, Pleasant Hill, and Franklin stations were estimated from the data at adjacent stations and were included in the calculation of averages.

An inspection of the table shows that the runoff in the Miami Valley is, on the average, about one-third of the rainfall. The runoff from the Buck Creek drainage area seems to be somewhat

less than in the other parts of the Mad River Valley. The total runoff from the Mad River drainage area, as shown by the records at the Wright station, is the same as the total runoff from the Stillwater River Valley, as shown by the West Milton records. The runoff in the upper Miami Valley seems to be practically the same as the runoff in the Mad and Stillwater drainage areas. The runoff in the Twin, Seven Mile, and Four Mile Creek areas, southwest of Dayton, seems to be higher than in the other parts of the valley. However, the records at these stations are somewhat more uncertain than those at the other stations, due to the greater difficulties encountered in obtaining the stream flow data; and it is doubtful if the runoff is actually much different from that of the other parts of the Miami Valley. The records at Hamilton, which are very satisfactory, seem to bear out this conclusion since they agree substantially with the Dayton records.

The runoff during the year 1916 was comparatively high, and the evaporation comparatively low, due to the large amount of storm rainfall that fell during the months of January, February, and March, when the available surface and ground storage was a minimum and the evaporation rate insignificant.

Records Above Hamilton

Table 22 gives the annual rainfall, runoff, evaporation, and ratio of runoff to rainfall, for the Hamilton station. Averages of the various quantities and departures from the averages are also given; and the maximum and minimum records are set in bold face type.

The rainfall records are accurate throughout. The maximum error for a single year probably does not exceed two per cent. The runoff and evaporation records are believed to be fairly accurate for all years except 1912. The runoff of 15.6 inches given for 1912 is believed to be considerably too low, inasmuch as the record for Dayton, for the same year is 23.1 inches. The Dayton record is probably too high. There seems to be no reason why the amounts at these two stations should be so different. The records for the years 1915 to 1919, inclusive, agree very well, as previously mentioned. The rainfall during the year 1912 was about the same as in 1913 but was much more uniformly distributed throughout the year. Consequently the runoff would be expected to be greater than normal but less than in 1913. Probably the average of the two records, 19.3 inches,

Table 22.—Annual Rainfall, Runoff, and Evaporation in the Miami Valley above Hamilton

Year Ending September 30	Rainfall			Runoff			Ratio of Runoff to Rainfall per cent	Evaporation		
	Inches	Per cent of Average	Departure from Average in per cent	Inches	Per cent of Average	Departure from Average in per cent		Inches	Per cent of Average	Departure from Average in per cent
1911	39.6	102.1	+2.1	13.5	95.9	—4.1	34.1	26.1	105.6	+5.6
1912	43.4	111.9	+11.9	15.6	110.8	+10.8	35.9	27.8	112.5	+12.5
1913	43.2	111.4	+11.4	24.4	173.3	+73.3	56.4	18.8	76.1	—23.9
1914	32.6	84.0	—16.0	10.5	74.6	—25.4	32.2	22.1	89.4	—10.6
1915	41.7	107.5	+7.5	11.2	79.6	—20.4	26.9	30.5	123.4	+23.4
1916	39.8	102.6	+2.6	18.8	133.5	+33.5	47.2	21.0	85.0	—15.0
1917	36.4	93.8	—6.2	11.6	82.4	—17.6	31.9	24.8	100.4	+0.4
1918	37.9	97.7	—2.3	9.6	68.2	—31.8	25.3	28.3	114.5	+14.5
1919	34.5	89.0	—11.0	11.5	81.7	—18.3	33.3	23.0	93.1	— 6.9
Average	38.79	100.0	7.89	14.08	100.0	26.13	36.3	24.71	100.0	12.53

is about the true value. Assuming this figure to be correct the evaporation for 1912 would be reduced from 27.8 inches to 24.1 inches.

Owing to the insufficiency of the stream flow records, the runoff for the year 1913 has been assumed to be the same as at Dayton, an assumption probably not much in error since the runoff at Dayton was well determined.

While the period of record, only nine years, is too short to give very satisfactory information, the data seems to show that the runoff is the most variable quantity of the three and that the rainfall is the least variable. The average departure of the annual runoff from the mean value is seen to be 26.1 per cent; the average departure of the annual evaporation, 12.5 per cent; and the average departure of the annual rainfall, only 7.9 per cent. A considerably greater value for the average departure of annual rainfall was obtained for the Miami Valley above Dayton, where records for 25 years are available, as will be discussed later. The maximum annual rainfall is seen to be 1.12 times the mean and 1.33 times the minimum; the maximum annual runoff, 1.73 times the mean and 2.54 times the minimum; and the maximum annual evaporation, 1.23 times the mean and 1.62 times the minimum.

The average ratio of runoff to rainfall is seen to be 36.3 per cent, or slightly greater than one-third. The maximum value occurred during the year 1913, probably due to the memorable flood of that year. Although the ratio was unusually low in 1915, when the evaporation was a maximum due to the large amount of storm rainfall occurring during the summer months, the actual minimum value occurred in 1918, amounting to only 25.3 per cent.

Records Above Dayton

Table 23 gives the annual rainfall, runoff, evaporation, temperature, and ratio of runoff to rainfall, for the years 1894 to 1918, inclusive, at the Dayton station, 25 years in all. Averages of the various quantities and departures from the averages are given as in table 22; and the maximum and minimum values are set in bold face type. The ratios of the maximum quantities to the mean and minimum quantities are also included.

The rainfall and temperature records are fairly satisfactory throughout. The runoff and evaporation records are more reliable during the years 1905 to 1918, inclusive, than they are dur-

Table 23.—Annual Rainfall, Runoff, Evaporation, and Temperature over the drainage area above Dayton

Year Ending September 30	Rainfall			Runoff			Ratio of Runoff to Rainfall in per cent	Evaporation*			Temperature**		
	Inches	Per cent of Average	Departure from Average in per cent	Inches	Per cent of Average	Departure from Average in per cent		Inches	Per cent of Average	Departure from Average in per cent	Degrees F	Per cent of Average	Departure from Average in per cent
1894	30.7	82.8	—17.2	4.9	41.3	—58.7	16.0	25.8	102.4	+2.4	54.7	103.7	+ 3.7
1895	24.0	64.8	—35.2	3.7	31.2	—68.8	15.4	20.3	80.6	—19.4	53.0	100.5	+ 0.5
1896	46.2	124.6	+24.6	8.1	68.2	—31.8	17.5	38.1	151.2	+51.2	54.0	102.4	+ 2.4
1897	33.3	89.8	—10.2	12.8	107.8	+ 7.8	38.4	20.5	81.4	—18.6	53.4	101.1	+ 1.2
1898	44.3	119.5	+19.5	14.7	123.8	+23.8	33.2	29.6	117.4	+17.4	55.0	104.2	+ 4.2
1899	34.2	92.3	— 7.7	9.7	81.7	—18.3	28.4	24.5	97.2	— 2.8	53.2	100.8	+ 0.8
1900	35.1	94.7	— 5.3	6.6	55.6	—44.4	18.8	28.5	113.1	+13.1	54.4	103.1	+ 3.1
1901	30.1	81.2	—18.8	5.6	47.2	—52.8	18.6	24.5	97.2	— 2.8	53.4	101.2	+ 1.2
1902	31.6	85.2	—14.8	3.8	32.0	—68.0	12.0	27.8	110.3	+10.3	51.1	96.8	— 3.2
1903	37.1	100.1	+ 0.1	12.6	106.1	+ 6.1	34.0	24.5	97.2	— 2.8	53.6	101.6	+ 1.6
1904	39.1	105.5	+ 5.5	13.1	110.3	+10.3	33.5	26.0	103.2	+ 3.2	49.8	94.4	— 5.6
1905	38.5	103.9	+ 3.9	7.1	59.8	—40.2	18.4	31.4	124.6	+24.6	51.5	97.6	— 2.4
1906	33.2	89.6	—10.4	9.2	77.5	—22.5	27.7	24.0	95.2	— 4.8	52.9	100.2	+ 0.2
1907	43.1	116.3	+16.3	17.2	144.9	+44.9	39.9	25.9	102.8	+ 2.8	51.9	98.4	— 1.6
1908	37.7	101.7	+ 1.7	17.7	149.1	+49.1	46.9	20.0	79.4	—20.6	53.1	100.6	+ 0.6
1909	39.3	106.0	+ 6.0	13.1	110.3	+10.3	33.3	26.2	104.0	+ 4.0	53.2	100.8	+ 0.8
1910	36.3	97.9	— 2.1	15.1	127.2	+27.2	41.6	21.2	84.1	—15.9	52.3	99.1	— 0.9
1911	39.8	107.4	+ 7.4	13.9	117.1	+17.1	34.9	25.9	102.8	+ 2.8	53.7	101.7	+ 1.7
1912	43.8	118.2	+18.2	23.1	194.6	+94.6	52.7	20.7	82.2	—17.8	50.8	96.3	— 3.7
1913	42.9	115.7	+15.7	24.4	205.5	+105.5	56.9	18.5	73.4	—26.6	54.0	102.4	+ 2.4
1914	32.3	87.1	—12.9	8.3	69.9	—30.1	25.7	24.0	95.2	— 4.8	53.3	101.0	+ 1.0
1915	41.8	112.8	+12.8	12.1	101.9	+ 1.9	28.9	29.7	117.8	+17.8	52.1	98.7	— 1.3
1916	39.9	107.6	+ 7.6	19.2	161.7	+61.7	48.1	20.7	82.2	—17.8	53.2	100.8	+ 0.8
1917	35.7	96.3	— 3.7	11.4	96.0	— 4.0	31.9	24.3	96.4	— 3.6	51.1	96.8	— 3.2
1918	36.8	99.3	— 0.7	9.4	79.2	—20.8	25.5	27.4	108.7	+ 8.7	50.3	95.4	— 4.6
Average	37.07	100.0	11.13	11.87	100.0	36.83	32.04	25.20	100.0	12.66	52.76	100.0	2.11
Max.÷Ave.	1.25	.....	.....	2.05	.....	.....	.....	1.51	.....	.....	1.04	.....	.....
Max.÷Min.	1.92	.....	.....	6.60	.....	.....	.....	2.06	.....	.....	1.10	.....	.....

*Taken equal to the difference between the rainfall and the runoff.
**Determined from records of the Dayton Cooperative Station.

ing the earlier years, although the data for 1912 is probably in error, as previously mentioned.

Figure 27 shows the total runoff, rainfall, and evaporation, in inches, and the average temperature in degrees Fahrenheit, for each year; also the mean annual values. Figure 28 shows the annual departures of the various quantities.

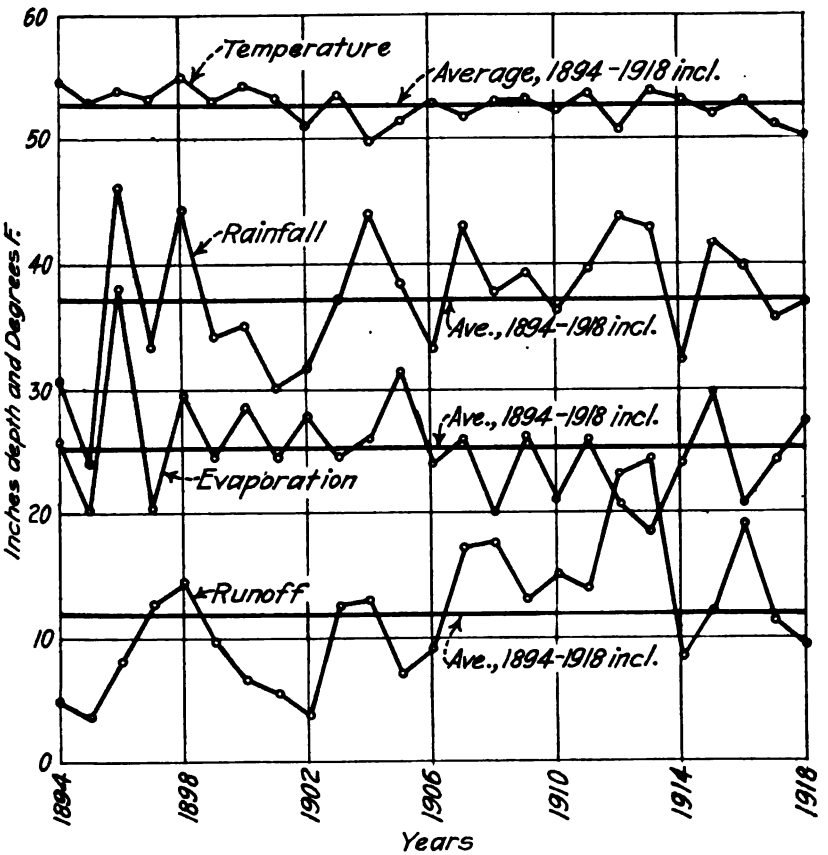


FIG. 27.—ANNUAL RAINFALL, RUNOFF, EVAPORATION, AND TEMPERATURE ABOVE DAYTON.

The horizontal lines indicate the mean annual values of the various quantities.

Table 23 shows that the mean annual rainfall is 37.07 inches; the mean annual runoff, 11.87 inches; the ratio of mean annual runoff to mean annual rainfall, 32.04 per cent, or slightly less than one-third; the mean annual evaporation, 25.20 inches; and the mean annual temperature, 52.76 degrees Fahrenheit. The

average departure of the annual rainfall is seen to be 11.13 per cent; of the runoff, 36.83 per cent; of the evaporation, 12.66 per cent; and of the temperature, 2.11 per cent. These figures show that the runoff is much more variable than either the rainfall or the evaporation, that the rainfall is only slightly less variable than the evaporation, a condition somewhat different from that shown by the 9-year record at Hamilton, and that the temperature is much less variable than any of the other quantities.

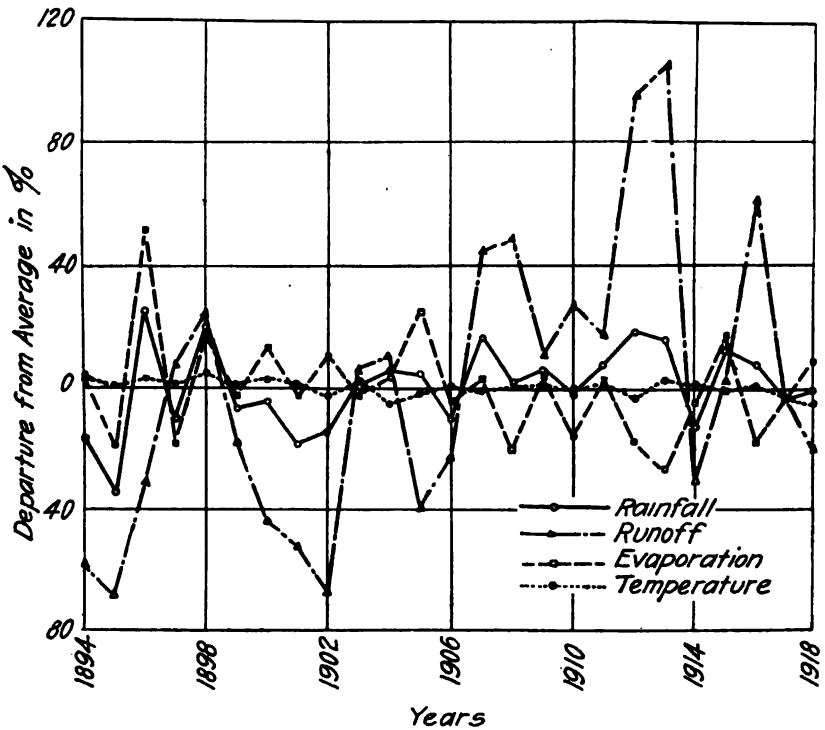


FIG. 28.—DEPARTURES OF ANNUAL RAINFALL, RUNOFF, EVAPORATION, AND TEMPERATURE ABOVE DAYTON.

The theory that the annual evaporation is a relatively constant quantity, which has been advanced by some engineers, does not appear to be true for the Miami Valley. The reason evaporation is variable is that the rainfall is variable. In order for the evaporation to be constant it would be necessary for the rainfall to be constant in distribution throughout the year as well as in quantity; inasmuch as the transpiration, which constitutes the greater part of the annual evaporation, varies with the amount of water available during the growing season. The variations

in annual evaporation probably decrease in importance as the proportion of the drainage area covered by water surfaces increases.

In figure 29 the runoff and evaporation departures have been platted as ordinates against the rainfall departures as abscissas. These diagrams seem to indicate that the runoff is generally

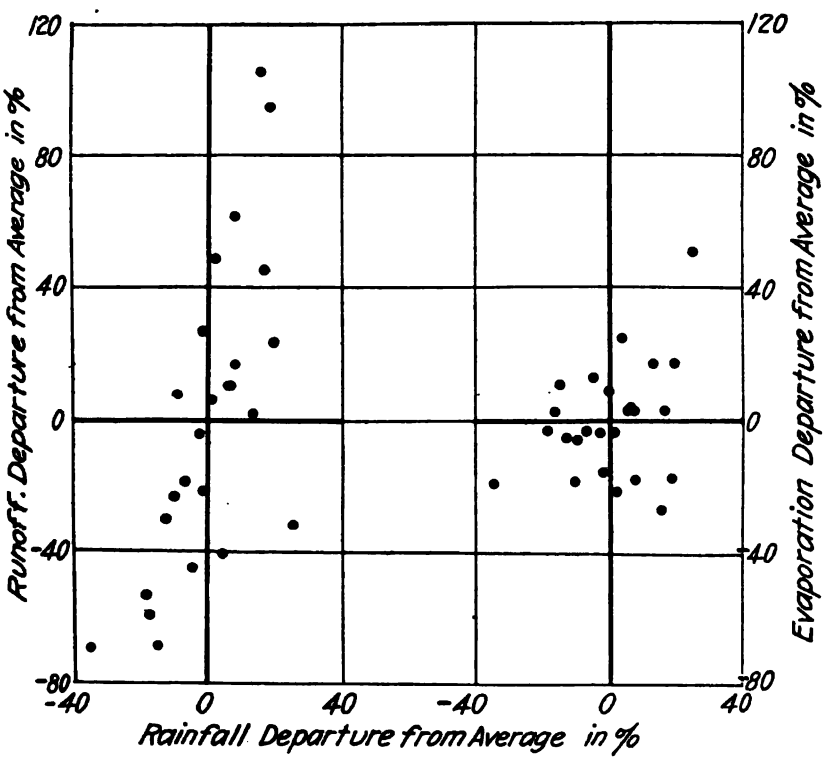


FIG. 29.—RELATIONS BETWEEN ANNUAL RAINFALL, RUNOFF, AND EVAPORATION ABOVE DAYTON.

Departures of annual runoff and evaporation are platted as ordinates against the departures of annual rainfall as abscissas.

greater than normal when the rainfall is greater than normal and vice versa; also that runoff is a more variable quantity than either rainfall or evaporation.

The maximum annual rainfall is seen to be 1.25 times the mean and 1.92 times the minimum; the maximum annual runoff, 2.05 times the mean and 6.60 times the minimum; the maximum annual evaporation, 1.51 times the mean and 2.06 times the min-

imum; and the maximum annual temperature, 1.04 times the mean and 1.10 times the minimum.

The minimum annual rainfall occurred during the year 1895, amounting to 24.0 inches, or about 65 per cent of the mean annual. The minimum annual runoff occurred during the same year and amounted to 3.7 inches, or about 31 per cent of the mean annual. While the runoff records for the years 1894 to 1904 are not so accurate as they are for the later years, the value of 3.7 inches in 1895 is checked, in a way, by the record of only 4.9 inches for the same year for the Muskingum drainage area above Zanesville, where the topography is slightly more rugged than in the Miami Valley.*

The maximum rainfall occurred in 1896, following the year of minimum rainfall, and amounted to 46.2 inches, or about 25 per cent more than normal. The runoff in 1896 was only 8.1 inches, or about 68 per cent of the normal amount; and the evaporation was 38.1 inches, or about 51 per cent more than normal, being the maximum for the entire record. However, this is an instance where the error due to the division into annual periods may have an appreciable value. Figure 30 shows the total rainfall on the drainage area above Dayton for each month of each year from 1894 to 1918, inclusive; and also the average for each month calculated from the 25 separate values. The rainfall during the months of July, August, and September, 1896, is seen to have been considerably greater than normal, amounting to 8.94, 5.06, and 6.13 inches, respectively. Consequently a part of this precipitation may have percolated through the surface soil; and, instead of being evaporated, appeared later, as ground water runoff. The study of monthly runoff, given later, shows that this delayed runoff probably did not exceed an inch. Assuming this figure to be correct the evaporation would be decreased to 37.1 inches in 1896, an amount still greater than any other annual amount, and increased to 21.5 inches in 1897; the runoff would be increased to 9.1 inches in 1896 and decreased to 11.8 inches in 1897; and the ratio of runoff to rainfall would be increased to 19.7 in 1896 and decreased to 35.4 in 1897. The unusual evaporation during the year 1896 was undoubtedly due to the large amount of storm rainfall that fell during the summer months when the plant requirements were a maximum.

The maximum runoff occurred in 1913, and amounted to 24.4 inches, or about 105 per cent more than normal. The minimum

*See U. S. Geological Survey Water Supply Paper 80, by George W. Rafter, 1903, page 85.

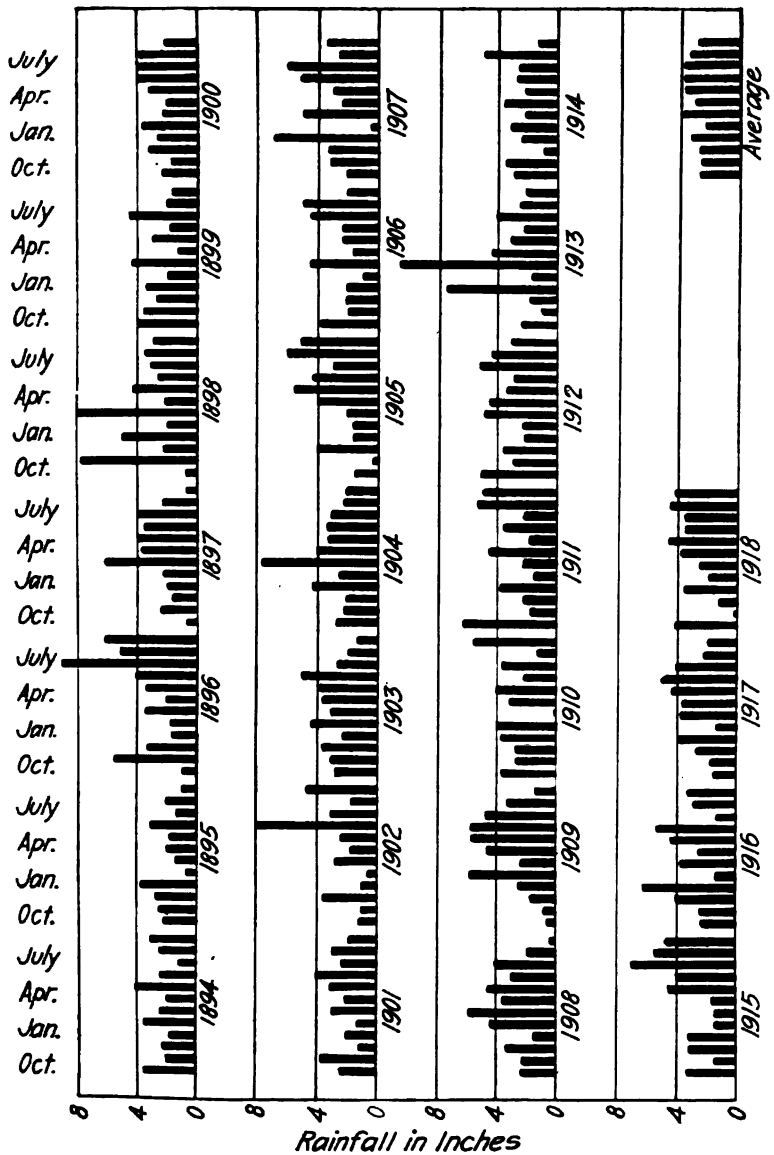


FIG. 30.—MONTHLY RAINFALL OVER THE DRAINAGE AREA ABOVE DAYTON. The mean monthly values based on the 25 years records are given in the lower right hand corner.

evaporation occurred during the same year, amounting to 18.5 inches or about 27 per cent less than normal. The large runoff and small evaporation in 1913 was undoubtedly due to the large amount of storm rainfall that fell during the months of January and March, when the conditions were most favorable for high rates of surface runoff.

A period of low rainfall and runoff occurred during the years 1899 to 1902, inclusive. During this time the amount of runoff gradually decreased until in 1902 it was only 3.8 inches, or only 0.1 of an inch greater than the minimum for the entire record. The minimum ratio of runoff to rainfall occurred during this year, amounting to only 12.0 per cent. In fact, a study of the data in table 23 seems to indicate that the ratio of runoff to rainfall is generally lower, as might be expected, during the dry periods than it is during the wet periods.

That variations in the amount of annual runoff and evaporation are due principally to the distribution of rainfall throughout the year rather than to the amount of the rainfall is indicated by the preceding discussion of conditions during the years 1896 and 1913. A study of figures 27, 28, and 30 leads to further corroboration of this conclusion. During the year 1914, when the rainfall was 32.3 inches, or about 13 per cent less than normal, the runoff was only 8.3 inches, or about 30 per cent less than normal, and the evaporation was 24.0 inches, or only about 5 per cent less than normal, due to the comparatively uniform distribution of the rainfall throughout the year. In 1915, the rainfall was 41.8 inches, or about 13 per cent more than normal; and was comparatively heavy during the summer months, when transpiration, surface and soil storage were comparatively great. Consequently the evaporation was 29.7 inches, or about 18 per cent more than normal, while the runoff was only 12.1 inches or about 2 per cent more than normal. In 1916, when the rainfall was 39.9 inches, or only about 8 per cent more than normal, the runoff was 19.2 inches, or about 62 per cent more than normal, and the evaporation was only 20.7 inches, or about 18 per cent less than normal, due to the large amount of precipitation that occurred as storm rainfall, principally during the winter months. The conditions during other years might be described but those already mentioned are probably sufficient to indicate the importance of rainfall distribution.

The three diagrams in figure 31 show the annual rainfall, runoff and evaporation departures, respectively, platted as ordi-

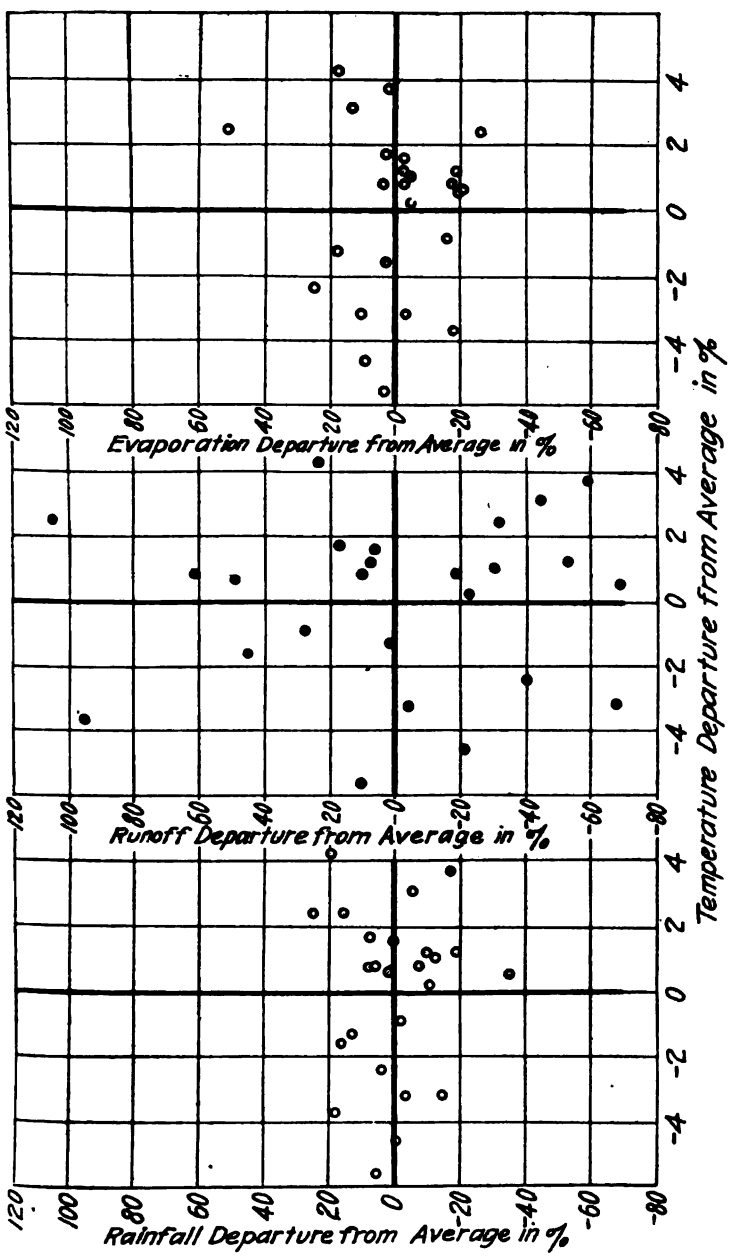


FIG. 31.—EFFECT OF TEMPERATURE ON ANNUAL RAINFALL, RUNOFF, AND EVAPORATION ABOVE DAYTON.

The departure of annual rainfall, runoff, and evaporation are platted as ordinates against the departure of annual temperature as abscissas.

nates against the annual temperature departures as abscissas. No definite relation seems to be shown by any of these diagrams. The variations in annual temperature are so small that whatever effect they may have on rainfall, runoff, or evaporation are not of sufficient magnitude to become noticeable.

SEASONAL RAINFALL AND RUNOFF

Table 24 gives the seasonal rainfall, runoff, retention, temperature, and ratio of runoff to rainfall for the drainage area of the Miami River above Dayton. The year is divided into three seasons, or periods, as they are generally termed; the replenishing, storage, and growing periods. The replenishing period includes the months of October, November, and December; the storage period, the months of January, February, March, and April; and the growing period, the months of May, June, July, August, and September.

Averages of the various quantities for the twenty-five years of record are included in the table; and the maximum and minimum values are set in bold face type. The ratios of the maximum to the mean and minimum quantities are also given. The data is shown graphically in figure 32.

The mean values given near the bottom of table 24 show that on the average the rainfall is about 7.69 inches, or about 21 per cent of the mean annual, during the replenishing period; about 12.23 inches, or 33 per cent of the mean annual, during the storage period; and about 17.13 inches, or 46 per cent of the mean annual, during the growing season. The average runoff appears to be about 1.69 inches, or 14 per cent of the mean annual, during the replenishing period; about 7.22 inches, or 61 per cent of the mean annual, during the storage period; and about 2.96 inches, or 25 per cent of the mean annual, during the growing period. The average retention appears to be about 6.00 inches, or 24 per cent of the mean annual, during the replenishing period; about 5.01 inches, or 20 per cent of the mean annual, during the storage period; and about 14.17 inches, or 56 per cent of the mean annual, during the growing season. The mean temperature is 43.1 degrees Fahrenheit, during the replenishing period, 38.2 degrees during the storage period, and 70.3 degrees during the growing period, the mean annual being 52.76 degrees, as previously mentioned. It is interesting to note the comparatively high retention and low runoff during the growing season

Table 24.—Seasonal Rainfall, Runoff, Retention, and Temperature above Dayton

Year Ending September 30	Replenishing Period					Storage Period					Growing Period				
	Rainfall in Inches	Runoff in Inches	Retention in Inches	Ratio Runoff Rainfall per cent	Temperature in Degrees F	Rainfall in Inches	Runoff in Inches	Retention in Inches	Ratio Runoff Rainfall per cent	Temperature in Degrees F	Rainfall in Inches	Runoff in Inches	Retention in Inches	Ratio Runoff Rainfall per cent	Temperature in Degrees F
1894	8.00	.82	7.18	10.2	43.5	9.62	2.67	6.95	27.7	**41.8**	13.05	1.43	11.62	11.0	71.6
1895	7.32	.76	6.56	10.4	42.0	7.60	1.88	5.72	24.7	37.0	**9.11**	**1.08**	**8.03**	11.8	72.5
1896	9.87	.87	9.00	8.8	42.8	8.96	3.04	5.92	33.9	40.4	**27.42**	4.16	**23.26**	15.2	71.7
1897	4.89	2.57	2.32	**52.6**	44.7	14.03	7.20	6.83	51.3	39.7	14.36	3.01	11.35	21.0	69.8
1898	10.74	1.65	**9.09**	15.4	46.2	17.08	10.23	6.85	59.9	40.9	16.46	2.82	13.64	17.1	71.5
1899	10.29	2.37	7.92	23.0	42.3	10.87	5.70	5.17	52.4	37.7	13.01	1.64	11.37	12.6	72.1
1900	7.26	.79	6.47	10.9	45.4	10.71	3.91	6.80	36.5	37.6	17.16	1.88	15.28	11.0	**73.2**
1901	7.49	1.15	6.34	15.4	**46.3**	8.30	2.83	5.47	34.1	36.1	14.32	1.67	12.65	11.7	71.5
1902	5.72	.56	5.16	9.8	40.4	**6.21**	**1.38**	4.83	**22.2**	36.1	19.68	1.81	17.87	**9.2**	69.7
1903	9.35	2.58	6.77	27.6	45.8	13.26	7.89	5.37	59.4	39.6	14.52	2.10	12.42	14.4	69.6
1904	6.91	.86	6.05	12.4	39.9	18.39	10.20	8.19	55.5	33.6	13.82	2.05	11.77	14.8	68.7
1905	5.61	.59	5.02	10.5	41.6	9.14	2.21	6.93	24.2	35.9	23.73	4.30	19.43	18.1	69.9
1906	7.82	1.78	6.04	22.7	42.9	9.20	5.59	3.61	60.8	37.6	16.15	1.78	14.37	11.0	71.2
1907	8.49	2.00	6.49	23.6	43.1	14.56	10.19	4.37	70.0	39.0	20.03	4.93	15.10	24.6	67.6
1908	8.00	2.12	5.88	26.5	41.6	15.47	11.88	3.59	76.8	39.6	14.19	3.72	10.47	26.2	70.8
1909	**3.16**	.91	**2.25**	28.8	44.5	15.33	6.34	**8.99**	41.3	40.3	20.82	5.83	14.99	**28.0**	68.8
1910	9.06	2.67	6.39	29.5	42.5	10.70	10.22	0.48	95.4	39.9	16.50	2.23	14.27	13.5	68.2
1911	10.18	4.19	5.99	41.1	40.9	11.98	7.06	4.92	58.9	40.3	17.69	2.67	15.02	15.1	72.3
1912	**11.43**	**5.63**	5.80	49.3	43.2	13.71	13.81	**—0.10**	**100.7**	**32.8**	18.70	3.64	15.06	19.5	69.8
1913	5.18	.89	4.29	17.2	44.6	**23.88**	**20.89**	2.99	87.5	39.8	13.82	2.58	11.24	18.7	70.9
1914	7.25	.82	6.43	11.3	45.6	11.27	5.93	5.34	**52.6**	36.9	13.64	1.56	12.08	11.4	71.0
1915	7.95	.94	7.01	11.8	43.5	7.74	5.06	2.68	65.4	39.0	25.53	**6.10**	19.43	23.9	67.7
1916	8.73	3.08	5.65	35.3	44.9	13.76	12.05	1.71	87.5	38.4	17.06	4.13	12.93	24.2	70.1
1917	6.07	.84	5.23	13.8	42.9	12.45	6.55	5.90	52.6	37.8	17.73	4.06	13.67	22.9	**66.6**
1918	5.53	.93	4.60	16.8	**36.6**	11.54	5.71	5.83	49.5	36.3	19.82	2.79	17.03	14.1	69.7
Average	7.69	1.69	6.00	22.0	43.1	12.23	7.22	5.01	59.1	38.2	17.13	2.96	14.17	17.3	70.3
Max. ÷ Ave.	1.49	3.33	1.51	.....	1.07	1.95	2.89	1.79	.....	1.10	1.60	2.06	1.65	.....	1.04
Max. ÷ Min.	3.62	10.0	4.04	.....	1.27	3.14	15.1	.....	.....	1.28	3.01	5.64	2.91	.....	1.10

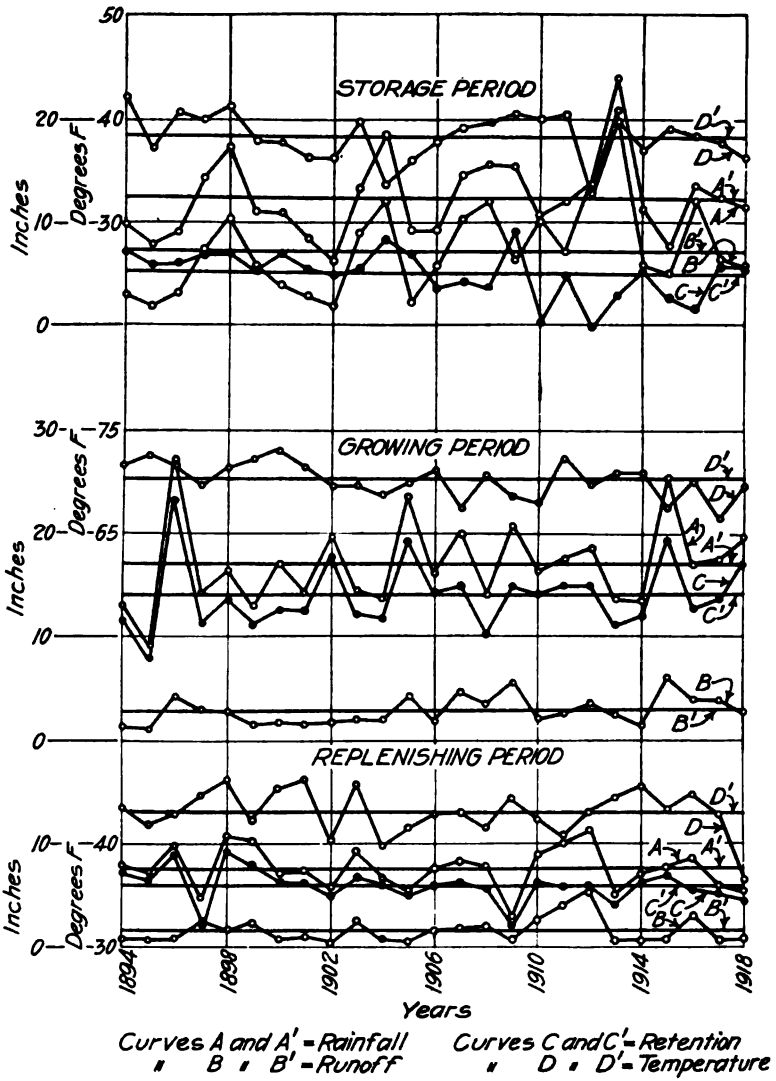


FIG. 32.—SEASONAL RAINFALL, RUNOFF, RETENTION, AND TEMPERATURE ABOVE DAYTON.

The storage period includes the months of January to April; the growing period, the months of May to September; and the replenishing period, the months of October to December. Curves A', B', C', and D' represent mean values for the 25 years of record.

and the comparatively low retention and high runoff during the storage season.

By making proper allowance for the drying out of the surface soil and for the maintenance of stream flow by ground water storage it is possible to estimate the total amount of evaporation during the growing period. The Moraine Park data, described in chapter III, indicates that the amount of moisture in the soil is reduced about 5 inches by transpiration and soil evaporation during the summer. If it is assumed that no percolation occurs during the growing period and that two-thirds of the stream flow during this time is surface runoff, assumptions which are believed to be reasonably correct for the Miami Valley, the evaporation during the growing season would be about 20 inches, or about 80 per cent of the mean annual evaporation. This would leave about 5 inches to be evaporated during the 7 months included in the replenishing and storage periods, corresponding to an average rate of about three-quarters of an inch per month which seems reasonable for the months of October to April, inclusive.

A comparison of the data in table 24 with that given in table 23 shows that the seasonal values are more variable than the annual values in practically all cases, as would naturally be expected. This is shown by the ratio of the maximum quantities to the mean and minimum values as well as by a study of the individual records. The ratios seem to indicate that the runoff during the growing season is slightly more constant than the total annual runoff. However, a study of the departures from the normal runoff for the growing period, not included herein, shows the annual runoff to be slightly more constant. The runoff seems to be more variable than either the rainfall or the retention, in all cases.

Replenishing Period

The maximum rainfall and the maximum runoff during the replenishing period, 11.43 and 5.63 inches respectively, both occurred in 1912, when the annual rainfall was considerably greater than normal. The maximum retention, amounting to 9.09 inches, occurred during the year 1898, the year of maximum annual temperature. The mean temperature during the replenishing period in 1898 was 46.2 degrees Fahrenheit, or only 0.1 of a degree less than the maximum value. The maximum temperature occurred in 1901. The minimum rainfall, 3.16 inches and

the minimum retention, 2.25 inches, both occurred in 1909. The minimum runoff, 0.56 inches occurred in 1902, when the total annual runoff was only 0.1 of an inch greater than the minimum annual amount. The minimum temperature for this period, 36.6 degrees, occurred in 1918, calendar year 1917, the early part of the unusually severe winter of 1917 and 1918. It is interesting to note that the minimum ratio of runoff to rainfall, for this period, occurred in the year 1896, immediately following the growing period in which the rainfall, runoff, and retention were all minimum values; also that the maximum value of this ratio occurred in 1897, immediately following the growing period in which the rainfall and retention were maximum values. The ratio was unusually low in 1896 because the necessary replenishing of the soil storage was considerably greater than usual. It was unusually high in 1897 because the replenishing had been accomplished to an extent greater than usual during the preceding period, thus causing a relatively high ground water flow.

Storage Period

The maximum values of rainfall and runoff during the storage period, 23.88 and 20.89 inches respectively, both occurred in 1913, the year of maximum annual runoff and minimum annual evaporation. These maxima were undoubtedly caused primarily by the great storm and resulting disastrous flood of March 23 to 27. The minimum rainfall, 6.21 inches, the minimum runoff, 1.38 inches, and the minimum ratio of runoff to rainfall, 22.2 per cent, all occurred in 1902, the year in which the ratio of annual runoff to annual rainfall was a minimum and in which the annual runoff was only 0.1 of an inch greater than the minimum value. The maximum retention, 8.99 inches, occurred in 1909. The minimum retention occurred in 1912, the runoff during this period being greater than the rainfall. This record, however, is probably uncertain due to errors in the runoff. As explained in the discussion of the records at Hamilton the runoff records for Dayton for 1912 are believed to be somewhat too high. The retention during the storage period was probably a minimum in 1910, when the record amounted to 0.48 of an inch. The maximum ratio of runoff to rainfall given as 100.7 in 1912 is also undoubtedly in error due to the error in runoff records. The value of 87.5 per cent given for the years 1913 and 1916 probably represents the maximum value for this period. The min-

imum temperature, 32.8 degrees, occurred in 1912; and the maximum, 41.8 degrees, occurred in 1894.

Growing Period

During the growing season the minimum rainfall, minimum runoff, and minimum retention all occurred in the year 1895, the year of minimum annual rainfall. The maximum rainfall and maximum retention occurred the following year, the year of maximum annual rainfall and maximum annual evaporation. The maximum runoff for this period occurred in 1915, and was probably due to the floods of July of that year. The minimum ratio of runoff to rainfall occurred in 1902, the year in which the minimum ratio for the storage period occurred, and in which the total annual runoff was only 0.1 of an inch greater than the minimum annual amount. The maximum value of this ratio occurred in 1909 when the runoff and rainfall were both considerably above normal. The maximum temperature occurred in 1900. The minimum temperature occurred in 1917 preceding the unusually severe winter of 1917 and 1918.

MONTHLY RAINFALL AND RUNOFF

Tables 25 to 28, inclusive, give the monthly rainfall, runoff, retention, and ratio of runoff to rainfall, for the drainage area of the Miami River above Dayton. Table 29 gives the monthly temperatures at Dayton. Maximum and minimum records are indicated as in the preceding tables; and the average values of the various quantities are included. Ratios of the maximum records to the mean and minimum records are given for the rainfall, runoff, retention, and temperature.

Figure 33 shows the maximum, mean, and minimum values of the various quantities. The maximum values are platted together in the upper part of the figure, the mean values near the center, and the minimum values in the lower part. It should be noted that since the maximum and minimum values of the rainfall, runoff, and retention frequently occur in different years, the relation that the retention is equal to the rainfall minus the runoff holds only for the mean curves. Only the mean values of the ratio of runoff to rainfall are platted. The maximum and minimum values are so erratic, due to the short periods of time considered, that they are practically meaningless. Figure 34 shows the same data as figure 33, arranged somewhat differently.

Table 25.—Average Monthly Rainfall on Drainage Area above Dayton for years Ending September 30, 1894 to 1918, inclusive, in Inches

Year Ending September 30	October	November	December	January	February	March	April	May	June	July	August	September	Total for Year
1894	3.62	2.10	2.28	1.76	3.51	2.36	1.99	4.07	2.38	1.12	2.42	3.06	30.67
1895	2.12	2.54	2.66	3.65	0.59	1.36	2.00	1.81	3.02	1.28	2.03	0.97	24.03
1896	0.99	5.44	3.44	1.70	1.76	3.44	2.06	3.37	3.92	8.94	5.06	6.13	46.25
1897	0.75	2.44	1.70	1.93	2.21	6.16	3.73	3.94	3.54	3.90	2.30	0.68	33.28
1898	0.79	7.63	2.32	4.98	1.99	7.87	2.24	4.28	2.62	3.14	3.55	2.87	44.28
1899	4.00	3.60	2.69	3.42	1.94	4.26	1.25	2.97	1.83	4.50	2.05	1.66	34.17
1900	2.28	1.81	3.17	2.74	3.69	2.29	1.99	3.30	3.98	3.95	3.81	2.12	35.13
1901	2.48	3.76	1.25	2.06	1.30	2.85	2.09	3.12	4.09	2.26	2.92	1.93	30.11
1902	1.16	0.96	3.60	1.00	0.65	2.82	1.74	2.45	7.92	3.01	1.64	4.66	31.61
1903	2.69	3.04	3.62	2.26	4.36	3.01	3.63	3.78	4.93	2.62	1.90	1.29	37.13
1904	2.68	2.15	2.08	4.19	2.50	7.70	4.00	3.22	3.31	3.00	2.22	2.07	39.12
1905	1.47	0.25	3.89	1.61	1.62	2.04	3.87	5.52	4.25	2.85	6.00	5.11	38.48
1906	3.83	1.90	2.09	2.09	0.94	4.47	1.70	2.34	2.40	4.50	4.88	2.03	33.17
1907	2.06	3.14	3.29	6.88	0.40	4.89	2.39	2.97	5.08	6.00	2.60	3.38	43.08
1908	2.34	2.26	3.40	1.51	4.52	5.89	3.55	4.55	3.05	4.13	2.05	0.41	37.66
1909	0.56	0.94	1.66	2.57	5.76	2.38	4.62	5.72	5.75	4.70	3.26	1.39	39.31
1910	3.61	2.72	2.73	3.62	3.93	0.07	3.08	4.00	2.10	3.56	1.30	5.54	36.26
1911	6.27	1.75	2.16	3.75	1.48	2.23	4.52	1.80	3.54	2.15	5.31	4.89	39.85
1912	5.02	2.92	3.49	2.14	2.27	4.81	4.49	3.35	2.86	5.16	4.30	3.03	43.84
1913	2.38	1.00	1.80	7.33	1.64	10.51	4.40	3.04	2.23	4.00	2.51	2.04	42.88
1914	2.92	3.43	0.90	2.40	3.08	2.25	3.54	2.19	2.66	2.63	4.88	1.28	32.16
1915	3.37	1.39	3.19	3.18	1.46	1.37	1.73	4.50	4.06	7.00	5.40	4.57	41.22
1916	2.35	2.40	3.98	6.20	1.26	3.73	2.57	4.33	5.26	1.41	2.92	3.14	39.55
1917	1.59	1.76	2.72	3.81	1.27	3.71	3.66	4.32	5.14	4.04	2.32	1.91	36.25
1918	4.01	0.31	1.21	3.46	1.86	2.55	3.67	4.48	3.42	3.44	4.42	4.06	36.89
Average	2.61	2.47	2.61	3.21	2.24	3.80	2.98	3.58	3.73	3.73	3.28	2.81	37.07
Max.÷Ave.	2.40	3.09	1.52	2.28	2.57	2.76	1.55	1.60	2.12	2.40	1.83	2.18	1.25
Max.÷Min.	11.2	30.5	4.42	7.33	14.4	150.	3.70	3.18	4.33	7.98	4.61	14.9	1.92

Table 26.—Monthly Runoff from Drainage Area above Dayton, in Inches, for Years Ending September 30, 1894 to 1918

Year Ending September 30	October	November	December	January	February	March	April	May	June	July	August	September	Total for Year
1894	.25	.24	.33	.33	1.11	.80	.43	.41	.32	.26	.22	.22	4.92
1895	.22	.24	.30	.70	.25	.57	.36	.26	.24	.21	.18	.19	3.72
1896	.16	.24	.47	.68	.74	1.03	.59	.40	.44	1.46	1.15	.71	8.08
1897	1.27	.57	.73	.61	1.55	3.56	1.48	1.10	.57	.70	.36	.28	12.78
1898	.21	.82	.62	2.94	.98	5.25	1.06	1.11	.50	.42	.43	.36	14.70
1899	.47	.78	1.12	2.30	.94	1.72	.74	.50	.34	.32	.25	.23	9.72
1900	.21	.22	.36	.86	.93	1.47	.65	.37	.51	.32	.47	.21	6.58
1901	.35	.42	.38	.51	.44	1.26	.62	.48	.51	.30	.20	.18	5.65
1902	.16	.14	.26	.24	.25	.49	.40	.24	.48	.67	.24	.18	3.76
1903	.64	.48	1.46	1.40	2.30	3.06	1.13	.56	.80	.31	.23	.20	12.56
1904	.23	.26	.37	1.94	2.21	3.74	2.31	.48	.51	.68	.20	.18	13.09
1905	.17	.16	.26	.26	.45	.83	.67	1.82	.64	.29	.56	.99	7.08
1906	.54	.52	.72	1.41	.58	2.16	1.44	.38	.26	.45	.48	.21	9.18
1907	.25	.78	.97	5.21	.94	3.28	.76	.64	1.95	1.40	.44	.50	17.15
1908	.48	.52	1.12	1.28	3.36	5.74	1.50	2.14	.57	.44	.38	.19	17.72
1909	.19	.23	.49	.64	2.62	1.66	1.42	3.11	1.00	.89	.56	.27	13.12
1910	.49	.76	1.42	4.66	2.41	2.59	.56	.75	.35	.42	.20	.51	15.13
1911	2.94	.44	.81	2.90	1.44	.86	1.86	1.11	.36	.29	.33	.58	13.91
1912	1.86	1.47	2.30	2.26	3.46	5.13	2.96	1.10	.72	.94	.51	.37	23.09
1913	.31	.29	.29	5.27	.94	10.51	4.17	1.09	.69	.37	.21	.22	24.36
1914	.23	.31	.28	.33	1.07	2.18	2.35	.68	.24	.21	.21	.22	8.33
1915	.34	.19	.41	.87	3.10	.66	.43	.40	.86	2.68	1.04	1.12	12.08
1916	1.01	.76	1.31	5.28	2.37	2.97	1.43	1.85	1.36	.36	.27	.29	19.25
1917	.23	.20	.41	1.79	1.24	2.32	1.20	1.14	1.02	1.40	.30	.20	11.43
1918	.29	.33	.31	.78	2.75	1.60	.58	1.24	.33	.31	.31	.60	9.42
Average	0.54	0.45	0.70	1.82	1.54	2.62	1.24	0.93	0.62	0.64	0.39	0.37	11.87
Max.÷Ave.	5.44	3.27	3.28	2.90	2.25	4.01	3.36	3.34	3.15	4.19	2.95	3.03	2.05
Max.÷Min.	18.4	10.5	8.84	22.0	13.8	21.5	11.6	12.9	8.13	12.7	6.40	6.22	6.60

Table 27.—Monthly Retention on Drainage Area Above Dayton for Years Ending September 30, 1894 to 1918, inclusive, in Inches

Year Ending September 30	October	November	December	January	February	March	April	May	June	July	August	September
1894	3.37	1.86	1.95	1.43	2.40	1.56	1.56	3.66	2.06	.86	2.20	2.84
1895	1.90	2.30	2.36	2.95	.34	.79	1.64	1.55	2.78	1.07	1.85	.78
1896	.83	5.20	2.97	1.02	1.02	2.41	1.47	2.97	3.48	7.48	3.91	5.42
1897	— .52	1.87	.97	1.32	.66	2.60	2.25	2.84	2.97	3.20	1.94	.40
1898	.58	6.81	1.70	2.04	1.01	2.62	1.18	3.17	2.12	2.72	3.12	2.51
1899	3.53	2.82	1.57	1.12	1.00	2.54	.51	2.47	1.49	4.18	1.80	1.43
1900	2.07	1.59	2.81	1.88	2.76	.82	1.34	2.93	3.47	3.63	3.34	1.91
1901	2.13	3.34	.87	1.55	.86	1.59	1.47	2.64	3.58	1.96	2.72	1.75
1902	1.00	0.82	3.34	.76	.40	2.33	1.34	2.21	7.44	2.34	1.40	4.48
1903	2.05	2.56	2.16	.86	2.06	— .05	2.50	3.22	4.13	2.31	1.67	1.09
1904	2.45	1.89	1.71	2.25	.29	3.96	1.69	2.74	2.80	2.32	2.02	1.89
1905	1.30	.09	3.63	1.35	1.17	1.21	3.20	3.70	3.61	2.56	5.44	4.12
1906	3.29	1.38	1.37	.68	.36	2.31	.26	1.96	2.14	4.05	4.40	1.82
1907	1.81	2.36	2.32	1.67	— .54	1.61	1.63	2.33	3.13	4.60	2.16	2.88
1908	1.86	1.74	2.28	.23	1.16	.15	2.05	2.41	2.48	3.69	1.67	.22
1909	.37	.71	1.17	1.93	3.14	.72	3.20	2.61	4.75	3.81	2.70	1.12
1910	3.12	1.96	1.31	—1.04	1.52	—2.52	2.52	3.25	1.75	3.14	1.10	5.03
1911	3.33	1.31	1.35	.85	.04	1.37	2.66	.69	3.18	1.86	4.98	4.31
1912	3.16	1.45	1.19	— .12	—1.19	— .32	1.53	2.25	2.14	4.22	3.79	2.66
1913	2.07	.71	1.51	2.06	.70	0	.23	1.95	1.54	3.63	2.30	1.82
1914	2.69	3.12	.62	2.07	2.01	.07	1.19	1.51	2.42	2.42	4.67	1.06
1915	3.03	1.20	2.78	2.31	—1.64	.71	1.30	4.10	3.20	4.32	4.36	3.45
1916	1.34	1.64	2.67	.92	—1.12	.76	1.14	2.48	3.90	1.05	2.65	2.85
1917	1.36	1.56	2.31	2.02	.03	1.39	2.46	3.18	4.12	2.64	2.02	1.71
1918	3.72	— .02	.90	2.68	— .89	.95	3.09	3.24	3.09	3.13	4.11	3.46
Average*	2.07	2.02	1.91	1.39	.70	1.18	1.74	2.65	3.11	3.09	2.89	2.44
Max.÷Ave.	1.80	3.37	1.90	2.12	4.48	3.35	1.84	1.55	2.39	2.42	1.88	2.22
Max.÷Min.	.....	.....	5.86	.....	.....	.....	13.9	5.94	4.99	8.70	4.94	24.6

*Computed from average values of monthly rainfall and runoff.

Table 28.—Ratio of Monthly Runoff to Monthly Rainfall on Drainage Area Above Dayton for Years Ending September 30, 1894, to 1918, inclusive, in Per Cent

Year Ending September 30	October	November	December	January	February	March	April	May	June	July	August	September
1894	**6.9**	11.4	14.5	18.8	31.6	33.9	21.6	10.1	13.4	23.2	9.1	7.2
1895	10.4	9.4	11.3	19.2	42.4	41.9	18.0	14.4	8.0	16.4	8.9	19.6
1896	16.2	4.4	13.7	40.0	42.0	30.0	28.6	11.9	11.2	16.3	**22.7**	11.6
1897	**169.5**	23.4	42.9	31.6	70.2	57.8	39.7	27.9	16.1	18.0	15.6	41.1
1898	26.6	10.8	26.7	59.0	49.2	66.8	47.3	25.9	19.1	13.4	12.1	12.6
1899	11.7	21.7	41.7	67.2	48.5	40.4	59.2	16.9	18.6	**7.1**	12.2	13.9
1900	9.2	12.2	11.4	31.4	**25.2**	64.2	32.7	11.2	12.8	8.1	12.3	9.9
1901	14.1	11.2	30.4	24.7	33.8	44.2	29.7	15.4	12.5	13.3	6.8	9.3
1902	13.8	14.6	7.2	24.0	38.5	17.4	23.0	9.8	**6.1**	22.3	14.6	**3.9**
1903	23.8	15.8	40.3	62.0	52.8	101.6	31.1	14.8	16.2	11.8	12.1	15.5
1904	8.6	12.1	17.8	46.3	88.4	48.6	57.8	14.9	15.4	22.7	9.0	8.7
1905	11.6	64.0	**6.7**	16.2	27.8	40.7	17.3	32.8	15.1	10.2	9.3	19.4
1906	14.1	27.4	34.5	67.5	61.7	48.3	84.7	16.2	10.8	10.0	9.8	10.3
1907	12.1	24.8	29.5	75.8	**235.0**	67.1	31.8	21.6	**38.4**	23.3	16.9	14.8
1908	20.5	23.0	32.9	84.8	74.4	97.5	42.3	47.1	18.7	10.6	18.5	**46.3**
1909	33.9	24.5	29.5	24.9	45.5	69.8	30.7	54.4	17.4	19.0	17.2	19.4
1910	13.6	28.0	52.0	**128.8**	61.4	**3700.0**	18.2	18.8	16.7	11.8	15.4	9.2
1911	46.9	25.2	37.5	77.3	97.3	38.6	41.2	**61.6**	10.2	13.5	6.2	11.9
1912	37.0	50.4	**65.9**	105.6	152.5	106.6	66.0	32.8	25.2	18.2	11.9	12.2
1913	13.0	29.0	16.1	71.9	57.3	100.0	**94.8**	35.9	30.9	9.2	8.4	10.8
1914	7.9	9.0	31.1	**13.8**	34.7	97.0	66.4	31.1	9.0	8.0	**4.3**	17.2
1915	10.1	13.7	12.9	27.4	212.2	48.2	24.8	**8.9**	21.2	**38.3**	19.3	24.4
1916	42.9	31.7	32.9	85.2	188.0	79.6	55.6	42.7	25.9	25.5	9.2	9.2
1917	14.5	11.4	15.1	47.0	97.6	62.6	32.8	26.4	19.8	34.7	12.9	10.5
1918	7.2	**106.2**	25.6	22.5	147.8	62.8	**15.8**	27.7	9.6	9.0	7.0	14.8
Average†	20.7	18.4	26.8	56.6	68.7	68.8	41.7	26.1	16.7	17.2	11.8	13.1

†Computed from average values of runoff and rainfall given in preceding tables.

Table 29.—Mean Monthly Temperature at Dayton, in Degrees F.

Year Ending September 30	October	November	December	January	February	March	April	May	June	July	August	September	Total for Year
1893	54.2	38.3	31.3	21.7	32.3	42.7	52.7	61.6	72.6	78.2	72.5	67.8	52.1
1894	54.4	40.9	35.3	35.8	30.7	47.4	53.3	62.5	74.4	76.7	74.3	70.3	54.7
1895	51.0	40.7	34.2	†29.6	†30.7	33.8	53.9	63.5	75.1	73.7	76.5	72.2	53.0
1896	48.8	42.6	37.0	32.0	33.8	35.6	60.0	71.2	72.0	76.5	74.4	64.4	54.0
1897	51.0	46.8	36.2	27.7	35.2	44.8	51.0	58.4	71.1	78.5	71.9	69.0	53.4
1898	60.4	44.2	34.0	34.0	33.0	47.1	49.4	64.0	75.4	74.2	75.2	70.6	55.0
1899	56.4	40.9	29.6	31.2	23.2	39.5	56.9	66.4	74.2	76.0	76.6	67.2	53.2
1900	59.4	45.7	31.2	33.8	28.5	35.6	52.4	65.2	72.4	76.4	79.2	72.8	54.4
1901	63.0	43.0	33.0	30.2	24.4	41.9	48.0	60.4	73.8	80.6	75.8	66.7	53.4
1902	54.8	38.4	28.0	28.9	22.2	43.0	50.2	65.4	69.8	75.8	71.9	65.4	51.1
1903	56.6	49.8	30.9	28.1	31.9	47.6	51.0	65.2	65.7	75.2	74.3	67.4	53.6
1904	55.6	38.2	25.8	21.3	25.3	42.2	45.4	61.6	70.2	73.6	71.6	66.6	49.8
1905	53.6	41.8	29.3	24.4	22.1	45.8	51.2	62.3	71.8	74.8	74.0	66.8	51.5
1906	53.8	41.2	33.6	36.0	28.4	32.3	53.8	63.2	71.3	74.0	76.6	70.8	52.9
1907	53.8	41.9	33.5	34.7	29.0	49.4	42.8	56.2	67.9	75.0	71.6	67.2	51.9
1908	50.6	39.8	34.3	31.0	30.2	44.8	52.2	64 2	70.8	75.8	73.8	69.2	53.1
1909	54.9	43.3	35.2	33.0	37.7	39.8	50.7	50.3	72.6	* 72.2	74.4	64.3	53.2
1910	50.0	51.0	26.4	29.7	27.0	50.4	52.6	57.2	68.2	75.0	73.0	67.7	52.3
1911	57.6	37.6	27.4	35.1	36.8	40.0	49.3	67.6	72.9	76.4	74.4	70.0	53.7
1912	54.6	37.7	37.4	19.0	23.6	34.0	54.4	64.7	68.9	75.4	71.6	68.4	50.8
1913	55.2	43.4	35.2	37.6	28.8	42.0	51.0	62.6	72.2	77.2	76.0	66.4	54.0
1914	54.9	46.3	35.5	34.8	23.9	37.6	51.3	63.6	73.9	77.0	74.5	66.0	53.3
1915	58.4	43.8	28.2	27.2	37.6	34.6	56.8	59.8	68.6	73.4	68.3	68.4	52.1
1916	56.6	45.9	32.1	37.0	28.4	37.4	50.6	63.8	66.2	79.7	75.9	65.0	53.2
1917	53.8	44.2	30.8	30.6	28.4	41.9	50.4	55.3	68.3	73.5	72.6	63.2	51.1
1918	48.2	40.0	21.7	15.7	33.0	46.4	50.0	68.2	70.8	72.3	78.2	59.2	50.3
Average	54.7	42.6	31.8	30.0	29.5	41.4	51.6	62.9	71.1	75.7	74.2	67.4	52.7
Max.÷Ave.	1.15	1.20	1.18	1.25	1.28	1.22	1.16	1.13	1.06	1.06	1.07	1.08	1.04
Max.÷Min.	1.31	1.36	1.72	2.39	1.71	1.56	1.40	1.29	1.14	1.12	1.16	1.23	1.10

†Normal values; records missing.

*Preceding data copied from cooperative station records in the local office of the U. S. Weather Bureau. Following data copied from U. S. Monthly Weather Review and Climatological data.

In this case the three temperature curves are kept together, the three rainfall curves together, and so on.

The mean distribution of the rainfall, runoff, retention, and temperature throughout the year is shown by the group of curves in the center of figure 33, as well as by the average values given at the bottom of the tables. The minimum mean monthly rainfall occurs during the month of February and amounts to 2.24 inches. If the record were increased so as to correspond to a month of 30 days instead of 28 the amount would still be less than the record for November, the next lowest month. March seems to be the month of heaviest rainfall, the average for this month being 3.80 inches. However, the value of 3.73 inches obtained for the months of June and July is practically as great. There appears to be a decrease in rainfall during April, the average amount for this month being only 2.98 inches. It is interesting to note that a similar decrease during this month is shown by the majority of the diagrams for southwestern Ohio, published by the U. S. Weather Bureau, in Volume II of Bulletin W.* The rainfall is generally low during the months of October, November, and December; and generally high during the months of May, June, and July.

The distribution of runoff during the year is slightly different from the rainfall distribution, inasmuch as it is generally low during the summer months. However, the month of greatest runoff is the same as the month of greatest rainfall, the monthly runoff being a maximum during March, amounting to 2.62 inches. September is the month of lowest runoff, the average for this month being only 0.37 inches.

The curve of mean retention follows, in a way, the curve of mean temperature, being high in the summer and low in the winter. The minimum monthly retention occurs in February, the month of minimum rainfall, and amounts to only 0.70 inches. June is the month of maximum retention, the average for this month being 3.11 inches.

The curve of the average ratio of runoff to rainfall is just the reverse of the temperature curve, being high in the winter and low in the summer. The maximum ratio of monthly runoff to monthly rainfall occurs in March, being 68.8 per cent. The minimum ratio occurs in August, being 11.8 per cent.

A study of the maximum and minimum values, and of the ratios given at the bottom of the tables, shows that the various

*Summary of Climatological Data East of the Mississippi River, Bulletin W, Volume II, U. S. Weather Bureau, Washington, D. C., 1912.

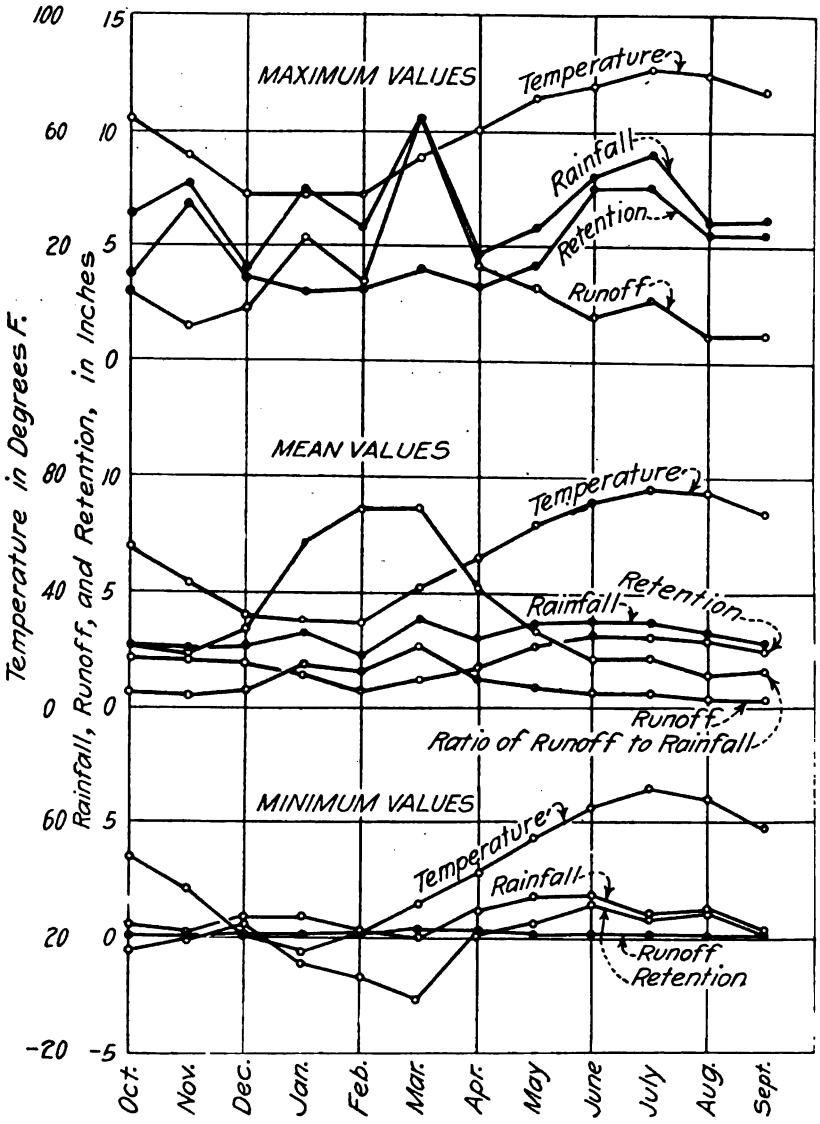


FIG. 33.—MAXIMUM, MEAN, AND MINIMUM MONTHLY RAINFALL, RUNOFF, RETENTION, AND TEMPERATURE ABOVE DAYTON.

The ratio of the mean monthly runoff to the mean monthly rainfall, expressed as a percentage, has been added to the group of mean values.

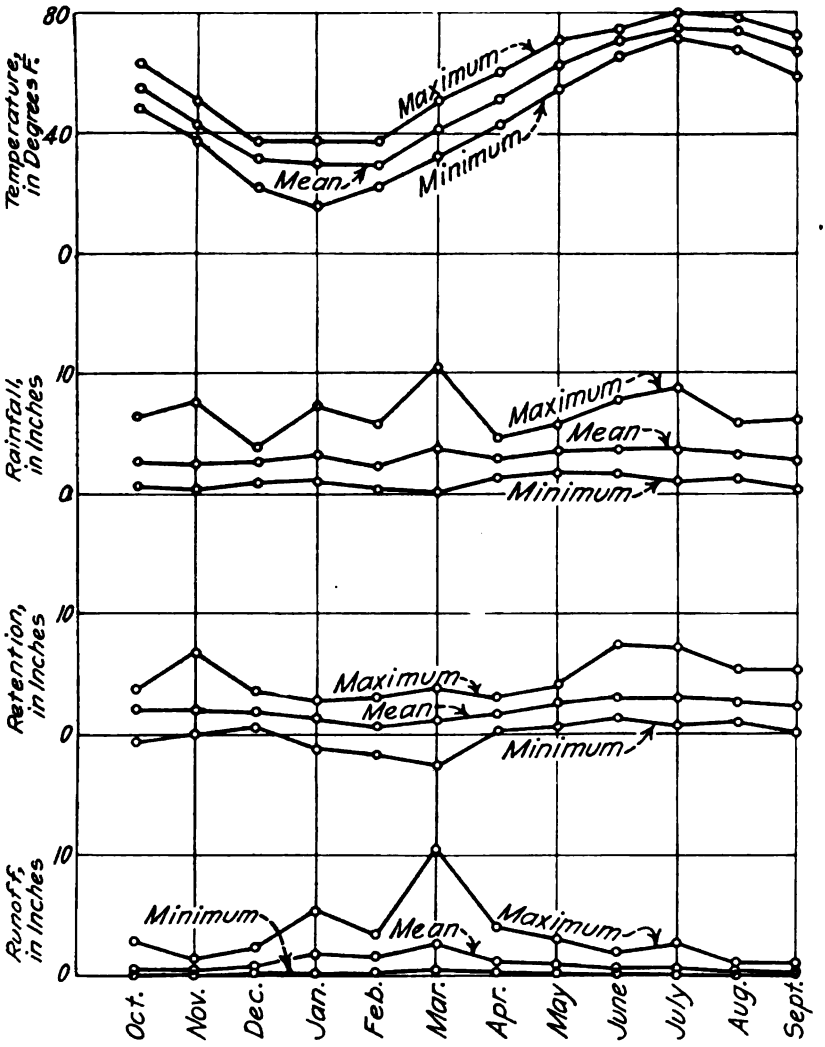


FIG. 34.—MAXIMUM, MEAN, AND MINIMUM MONTHLY RAINFALL, RUNOFF, RETENTION, AND TEMPERATURE ABOVE DAYTON.

Curves shown in figure 33 have been arranged so as to show more clearly the differences between the maximum and the minimum values of the various quantities.

quantities are all quite variable, much more variable than the annual or seasonal quantities, as would naturally be expected. The runoff again seems to be considerably more variable than either the rainfall or the retention.

The maximum rainfall during any one month of the entire record occurred in March, 1913, due to the great storm of March 24 to 27, and amounted to 10.51 inches. The maximum runoff for any one month also occurred during March, 1913, and amounted to the same value. The ground water flow at this time was unusually high due to the heavy precipitation of the preceding January. The minimum rainfall for the entire period occurred in March, 1910, and amounted to only 0.07 of an inch. The minimum runoff occurred in November, 1902, calendar year 1901, amounting to only 0.14 of an inch.

Maximum and minimum values of monthly retention and ratio of runoff to rainfall, as well as all individual values, are more or less erratic due to the short period of time considered. The runoff very frequently is not comparable with the rainfall for the same month. Probably the chief value of the data given in tables 27 and 28 is to show this. During the winter months the runoff is often greater than the precipitation, due either to snow accumulations or to floods in the early part of the month caused by heavy precipitation during the late part of the preceding month. The runoff during February, 1916, was greater than the rainfall, due to the flood runoff resulting from the heavy precipitation of January 30 and 31. The runoff of February, 1918, was greater than the rainfall, due to the melting of the heavy snows which had fallen during January. Other negative values of retention might be similarly explained.

The curves of maximum, mean, and minimum monthly temperature given in figure 34 need no discussion. However, it is interesting to note that the minimum records for October, December, and January all occurred during the unusually severe winter of 1917 and 1918.

SURFACE AND GROUND WATER FLOW

Table 30 shows the relation between surface and ground water flow for the drainage areas of the Miami River above Dayton, of Mad River above Wright, of Buck Creek above Springfield, and of Stillwater River above West Milton. The annual amounts of surface runoff and of ground water runoff are given in inches depth over the drainage areas and in percentages of the total

runoff, for each year for which records are available. The total annual runoff in inches is also included. The maximum and minimum records are indicated as before.

The proportions of annual runoff which appear as surface or flood flow and as low water or ground water flow can be determined only approximately. No exact separation is possible. In calculating the data given in table 30, the separation was made on the hydrographs in the following manner: Lines representing the rate of ground water flow were drawn so as to pass through the low points only, as shown in figures 35 to 39, inclusive. The endeavor was to draw the lines so that the increased flow of tiles immediately after a flood, that is, the drainage of the surface soil, would be included in the surface or flood runoff rather than in the ground water runoff, since such flow acts more nearly like surface flow than like low water flow. It was also assumed that no percolation occurs during the growing season or before the latter part of the replenishing period, that is, during the period from about May 1 to about December 1. Having arbitrarily drawn the curve representing the rate of ground water flow, it was, of course, simply a matter of calculation to determine the total amounts of surface and ground water runoff during the year.

Reference to table 30 shows that in the Miami Valley above Dayton the surface flow is about two-thirds of the total runoff, and the ground water flow about one-third. In the Buck Creek Valley the surface flow contributes only about 44 per cent of the total and the ground water flow about 56 per cent. In the Mad River Valley above Wright, including the Buck Creek Valley, the surface flow amounts to about 53 per cent of the total and the ground water flow, to about 47 per cent. In the Stillwater River Valley the surface flow constitutes about 79 per cent of the total and the ground water flow, only about 21 per cent.

These wide differences in the proportions of surface and ground water flow are the result of variations in geological and soil conditions. In the Mad River Valley there is relatively large underground storage in deep deposits of glacial gravel, while the comparatively loose and shallow surface soil permits rapid percolation. Gravel deposits are less extensive in the Miami Valley above Dayton, and still less frequent in the Stillwater Valley. Over a considerable portion of the latter basin there are but a few feet of residual clay soil overlying the bed rock, which generally is limestone. On these drainage areas surface slope

Table 30.—Surface and Ground Water Runoff in the Miami Valley

Year Ending September 30	Total Runoff in Inches	Surface Runoff		Ground Water Runoff	
		Inches	% of Total	Inches	% of Total
DRAINAGE AREA OF THE MIAMI RIVER ABOVE DAYTON					
1894	4.92	1.90	38.6	3.02	61.4
1895	3.72	1.29	34.7	2.43	65.3
1896	8.08	4.47	55.3	3.61	44.7
1897	12.78	8.19	64.1	4.59	35.9
1898	14.70	10.41	70.8	4.29	29.2
1899	9.72	5.32	54.7	4.40	45.3
1900	6.58	3.16	48.0	3.42	52.0
1901	5.65	2.67	47.3	2.98	52.7
1902	3.76	1.56	41.5	2.20	58.5
1903	12.56	7.77	61.9	4.79	38.1
1904	13.09	9.38	71.7	3.71	28.3
1905	7.08	4.46	63.0	2.62	37.0
1906	9.18	5.25	57.2	3.93	42.8
1907	17.16	11.38	66.3	5.78	33.7
1908	17.72	12.52	70.7	5.20	29.3
1909	13.12	8.31	63.3	4.81	36.7
1910	15.13	10.56	69.8	4.57	30.2
1911	13.91	9.18	66.0	4.73	34.0
1912	23.09	16.18	70.1	6.91	29.9
1913	24.36	19.71	80.9	4.65	19.1
1914	8.33	4.95	59.4	3.38	40.6
1915	12.09	8.58	71.0	3.51	29.0
1916	19.25	14.19	73.7	5.06	26.3
1917	11.43	7.41	64.8	4.02	35.2
1918	9.42	6.44	68.4	2.98	31.6
1919	11.15	6.71	60.2	4.44	39.8
Average	11.85	7.77	65.6	4.08	34.4
DRAINAGE AREA OF BUCK CREEK ABOVE SPRINGFIELD					
1915	8.34	3.92	47.0	4.42	53.0
1916	14.75	6.73	45.6	8.02	54.4
1917	10.25	4.77	46.5	5.48	53.5
1918	10.10	4.64	45.9	5.46	54.1
1919	11.04	4.10	37.1	6.94	62.9
Average	10.89	4.83	44.3	6.06	55.7
DRAINAGE AREA OF MAD RIVER ABOVE WRIGHT					
1915	12.03	6.86	57.0	5.17	43.0
1916	19.39	11.85	61.1	7.54	38.9
1917	13.48	6.75	50.1	6.73	49.9
1918	11.09	5.69	51.3	5.40	48.7
1919	12.89	5.56	43.1	7.33	56.9
Average	13.78	7.34	53.3	6.44	46.7
DRAINAGE AREA OF STILLWATER RIVER ABOVE WEST MILTON					
1915	12.63	10.63	84.2	2.00	15.8
1916	17.63	14.30	81.1	3.33	18.9
1917	12.33	8.73	70.8	3.60	29.2
1918	13.30	10.34	77.7	2.96	22.3
1919	12.00	9.48	79.0	2.52	21.0
Average	13.58	10.70	78.8	2.88	21.2

has but little influence on runoff. In fact the surface slopes are steeper over the Mad River drainage area where percolation is great, than on the Stillwater where flood runoff predominates.

Considering the Dayton records the surface flow is seen to vary from only 1.29 inches in 1895, the year of minimum annual runoff, to 19.71 inches in 1913, the year of maximum annual runoff. The percentages of the totals for these years were 34.7 and 80.9 respectively, which are the minimum and maximum percentages. The ground water flow varied from 2.20 inches in 1902, the year in which the rainfall during the storage period, when practically all of the percolation occurs, was a minimum, to 6.91 inches in 1912. However, as previously mentioned, the record for the year 1912 is believed to be too high. Probably the value of 5.78 inches given for the year 1907 represents the true maximum amount. The ground water flow was a maximum percentage of the total in 1895, the year of minimum surface and minimum total runoff; and a minimum percentage of the total in 1913, the year of maximum surface and maximum total runoff.

The minimum and maximum values for the other drainage areas are, of course, very uncertain due to the shortness of the record. However, a study of the averages of the Dayton records for the years 1915 to 1919 inclusive, and of those for the entire period of record, indicates that the averages for the other drainage areas are not greatly in error.

The annual ground water runoff is much less variable than the annual surface flow, as would be expected. The maximum value of the annual surface runoff in the Miami Valley above Dayton, 19.71 inches, is about 15.3 times the minimum value, while the maximum value of the ground water runoff, using the 1912 record, is only about 3.14 times the minimum value.

MASS CURVES

The hydrology of a drainage area may be shown conveniently by means of mass curves. Such curves have been drawn for the Mad River Valley above Wright for the years 1915 to 1919, inclusive. They are shown in figures 35 to 39, one year's records being shown in each figure. Separate curves have been drawn to show the rainfall, ground water runoff, flood runoff, total runoff, retention, soil absorption, percolation, and evaporation. In order to avoid confusion the rainfall curve was arbitrarily started at 10 inches on the scale instead of at 0. Hydrographs showing the rate of discharge and the arbitrary separation of

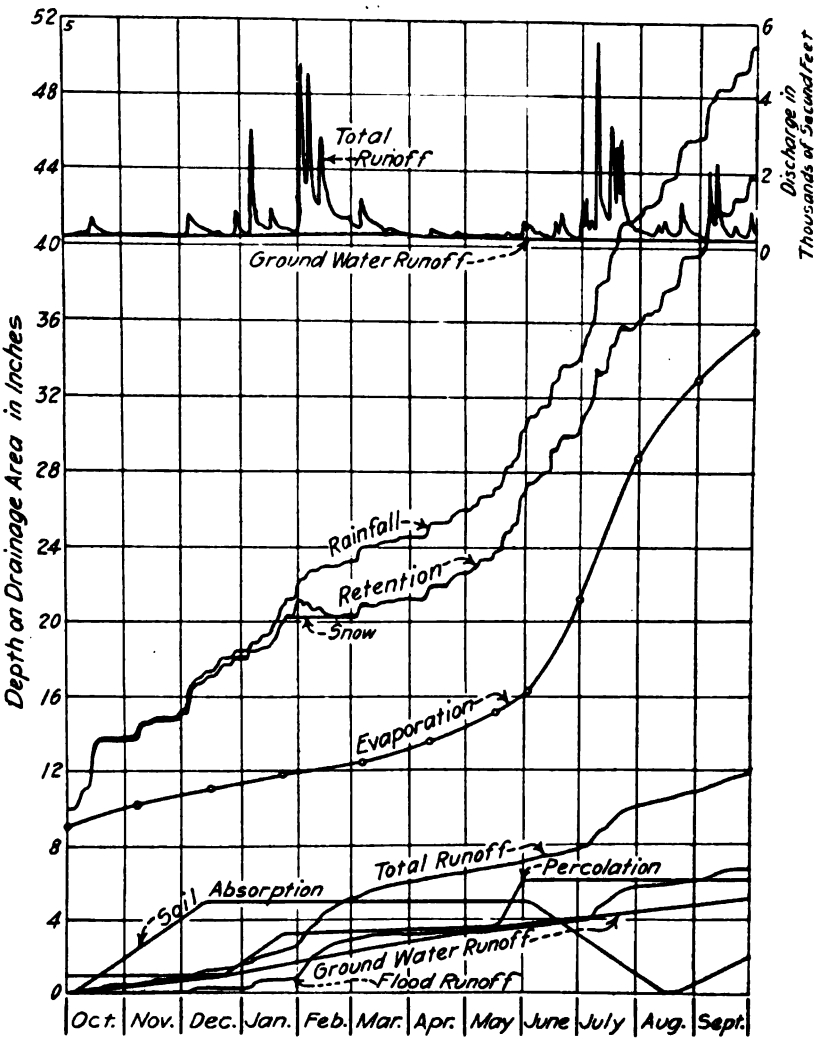


FIG. 35.—HYDROLOGY OF THE MAD RIVER VALLEY ABOVE WRIGHT DURING 1915

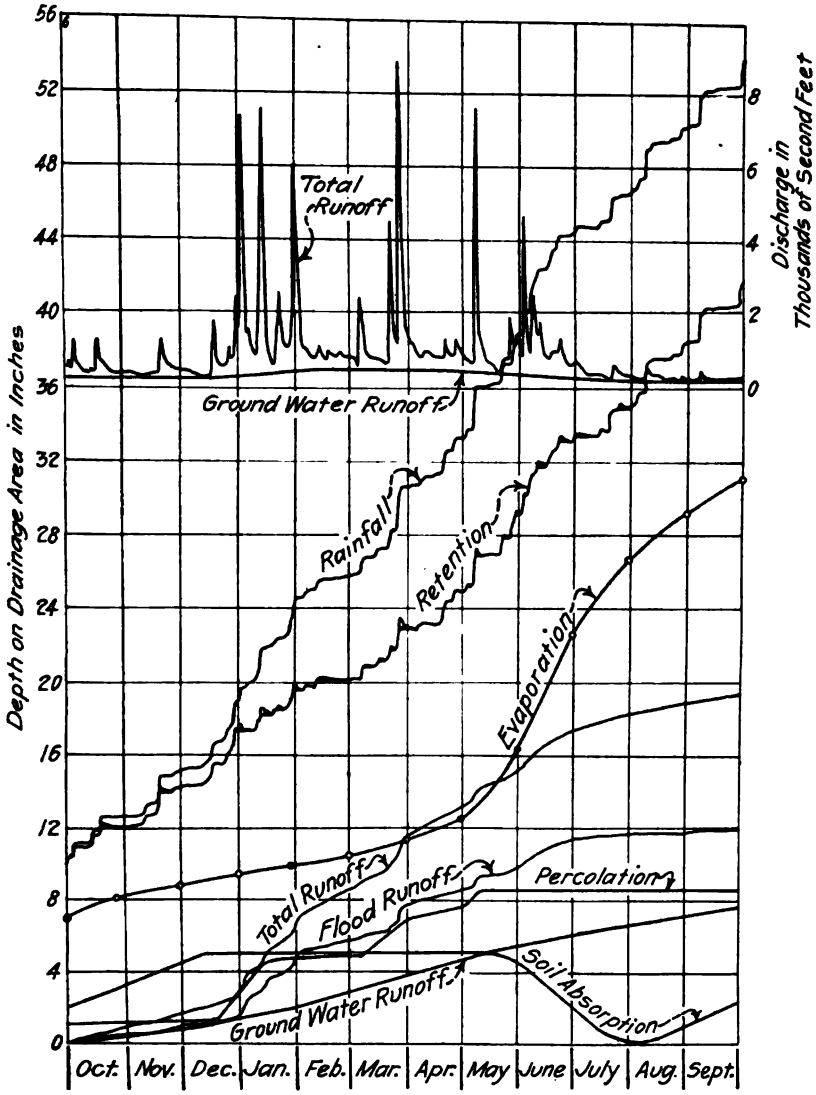


FIG. 36.—HYDROLOGY OF THE MAD RIVER VALLEY ABOVE WRIGHT DURING 1916.

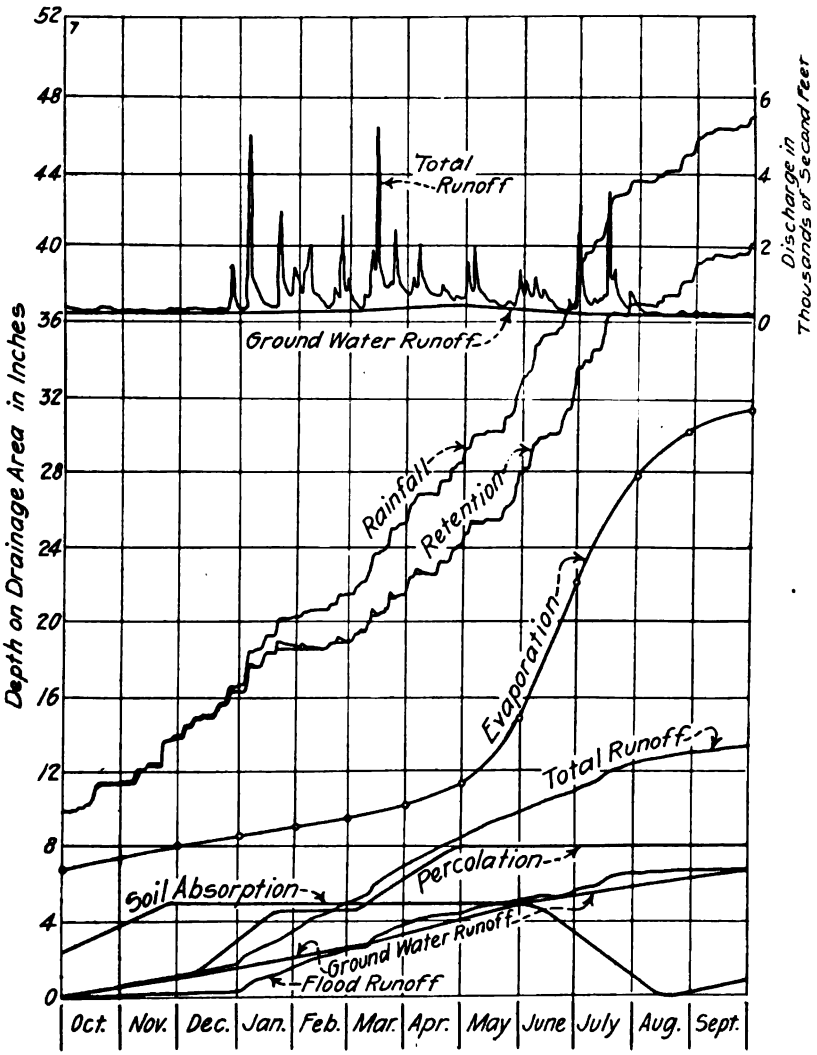


FIG. 37.—HYDROLOGY OF THE MAD RIVER VALLEY ABOVE WRIGHT DURING 1917.

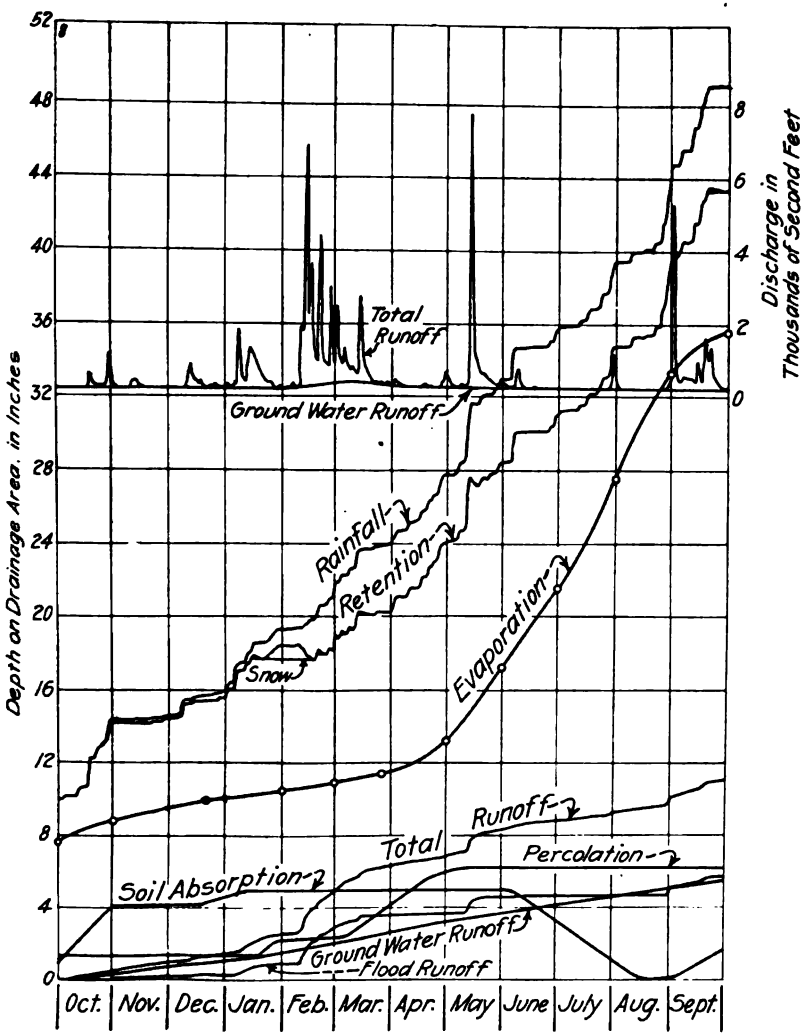


FIG. 38.—HYDROLOGY OF THE MAD RIVER VALLEY ABOVE WRIGHT DURING 1918.

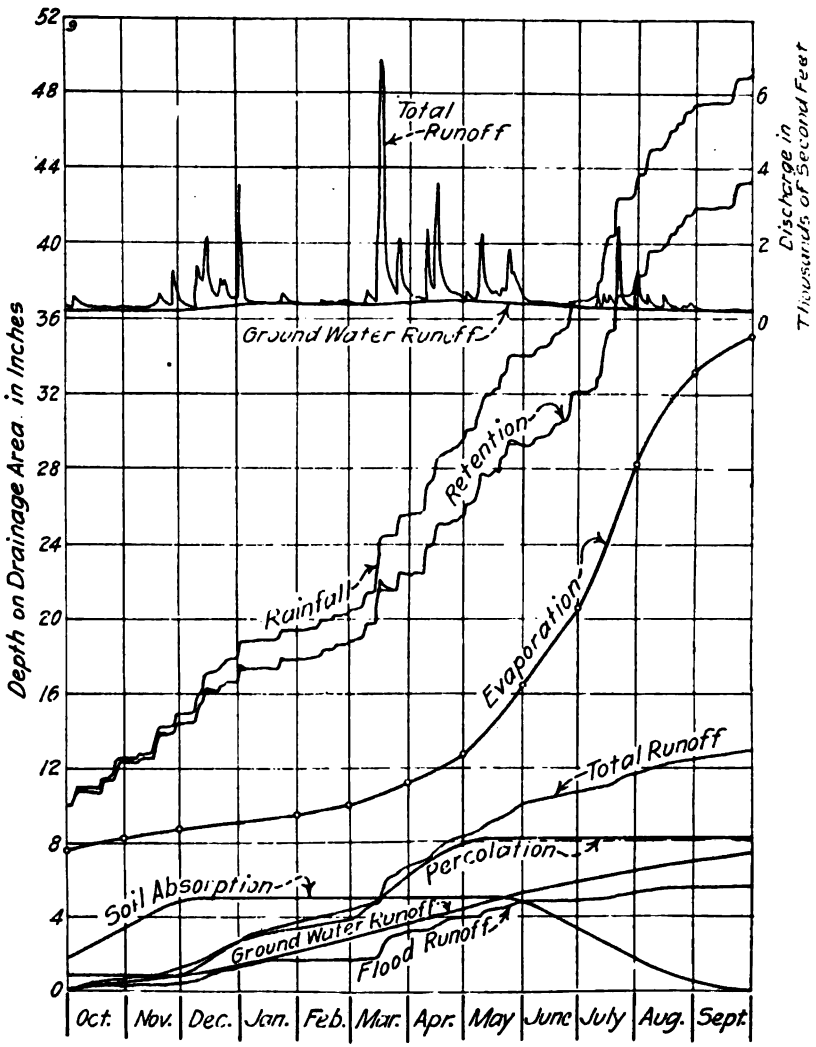


FIG. 39.—HYDROLOGY OF THE MAD RIVER VALLEY ABOVE WRIGHT DURING 1919.

flood runoff from ground water runoff are platted in the upper parts of the figures.

No explanation of the four curves mentioned first is needed. They were simply drawn from the rainfall and runoff data determined as previously described. The retention curve was obtained by subtracting the flood runoff from the rainfall. Horizontal lines have been drawn under the peaks, or humps, on the retention curves, thus indicating the surface storage. The larger humps, having a comparatively long duration and occurring during the winter months, are due to precipitation in the form of snow. The two larger humps due to this cause have been marked "snow", but the others have not been designated. The sharp peaks of comparatively short duration represent storage on the ground or in the stream channels during flood periods. The retention curve, as modified by the horizontal lines, represents the total of the soil absorption, percolation, and evaporation curves, the ground water runoff being maintained by the ground water storage or percolation water. The lines under the humps should really have been drawn so as to slope upward toward the right instead of horizontal, since soil absorption and evaporation are continuous, to some extent at least, throughout the storm period.

The soil absorption curves, or soil storage curves as they might be termed, were drawn after a careful study of the Moraine Park soil moisture records given in chapter III. It was assumed that there is a variation of five inches in the amount of moisture in the soil during the year; that the soil reaches its dryest condition sometime late in the summer, during August or September; that it gradually fills with moisture in the fall, during the months of September, October, November, and December; and that it then remains saturated until late in the spring, when it begins to dry out due to transpiration and increased soil evaporation.

In drawing the percolation curves it was assumed that no percolation occurs during the summer or early fall months; that is, that percolation ceases about the time the soil begins to dry out in the spring and does not begin until late in the fall, about the time the surface soil becomes saturated. It was also assumed that the percolation curve joins the ground water runoff curve at the time percolation begins. The former assumption is believed to be essentially correct for the Miami Valley except in very unusual instances. The latter assumption, while more

or less arbitrary, does not lead to appreciable error. Of course the percolation curves do not need to touch the ground water runoff curves at the time percolation begins. They could have been drawn a fixed distance above, at this time; that is, the curves shown in the figures could have been arbitrarily raised a certain amount.

By drawing the percolation curves in this way the total percolation during a given winter and spring was made just great enough to maintain the ground water flow until percolation began in the following fall or winter. It is not believed that in the Miami Valley percolation during a given storage period ever affects greatly the ground water flow after percolation begins in the succeeding fall; or, in other words, that the ground water level at the time percolation begins ever varies greatly from year to year. The minimum amount of annual ground water runoff in the Miami Valley above Dayton occurred in 1902, following a year in which the rainfall was only 30.1 inches, or about 7 inches less than normal. In 1914, when the rainfall was 32.3 inches, or about 5 inches less than normal, following a year in which the rainfall was 42.9 inches, or about 6 inches more than normal, the ground water runoff amounted to 3.38 inches, or about 1.18 inches more than in 1902. A part of this 1.18 inches was probably due to the 2 inches greater rainfall in 1914. It does not seem probable that percolation during a particularly wet season ever increases the ground water runoff during the following year by as much as an inch.

In drawing the percolation curves during the winter and spring when percolation was taking place, consideration was given to the rainfall distribution and form of occurrence as well as to the temperature and other meteorological conditions.

The evaporation curves were determined by subtracting from the retention curves, or from the horizontal lines under the retention curves, the sum of the soil absorption and percolation curves. In doing this points were taken about a month apart as shown on the diagrams. The attempt was to show the general shape of the evaporation curve throughout the year rather than the daily variations. It is only at the points indicated that the evaporation is equal to the retention less the sum of the soil absorption and percolation. In order for this relation to hold throughout it would be necessary to throw the small irregularities of the retention curves into the evaporation, absorption, or percolation curves. It is probable that during the summer and fall the ir-

regularities should be thrown into the evaporation and absorption curves; and that during the winter and spring they should be thrown into the evaporation and percolation curves. More irregularities would be expected in the evaporation curve during the summer than during the winter. Data on transpiration and soil evaporation seems to indicate that during the summer months rates of evaporation as great as a half an inch a day, or even greater, may occur immediately after a heavy rain. Such conditions would cause jumps in the evaporation curve somewhat similar to those in the retention curve.

Table 31 gives the monthly evaporation, taken from the curves, for the Mad River Valley above Wright for each year; and also the average amount for each month, based on the five years' records. The average monthly evaporation from a water

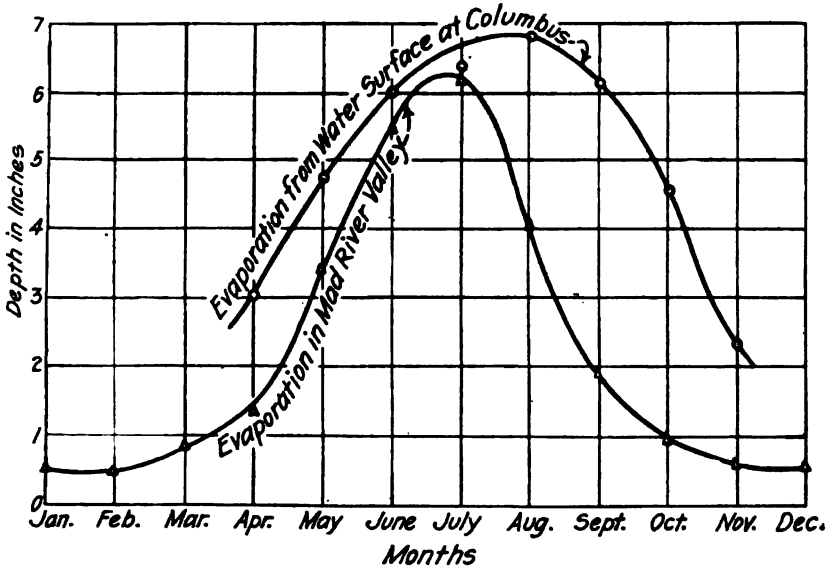


FIG. 40.—MONTHLY EVAPORATION IN THE MAD RIVER VALLEY ABOVE WRIGHT

For comparative purposes a curve has been added showing the monthly evaporation from water surface at Columbus.

surface at Columbus, for the years 1907 and 1908, for the months of April to November is also included.* Figure 40 shows graphically the data on average monthly evaporation given in table 31. It will be noticed that the evaporation from a water surface is

*Water Resources of Illinois by A. H. Horton, Report of Rivers and Lakes Commission, State of Illinois, Springfield, Illinois, 1914, page 310.

somewhat higher than the evaporation from the land, especially during the fall months. The evaporation from the land is comparatively low during the fall because the available supply of moisture has been depleted by the high rates of transpiration and soil evaporation during the growing season.

Table 31.—Monthly Evaporation on Drainage Area Above Wright Station, in inches, 1915 to 1919, inclusive

Month	Years					Average	Average Evaporation* from Water Surface
	1915	1916	1917	1918	1919		
October....	1.04	1.18	.63	1.20	.65	.94	4.58
November..	.67	.57	.57	.70	.48	.60	2.35
December..	.67	.60	.55	.53	.42	.55	.....
January....	.55	.55	.55	.52	.40	.51	.....
February...	.47	.48	.45	.40	.55	.47	.....
March.....	.80	.85	.75	.60	1.20	.84	.....
April.....	1.20	1.17	1.15	1.60	1.50	1.32	3.05
May.....	1.70	3.90	3.60	4.15	3.70	3.41	4.73
June.....	5.15	6.30	7.20	4.30	4.25	5.44	5.99
July.....	7.60	4.05	5.65	5.90	7.70	6.18	6.37
August.....	4.15	2.60	2.65	5.95	4.95	4.06	6.81
September..	2.60	1.90	.95	2.15	1.80	1.88	6.11
Total.....	26.60	24.15	24.70	28.00	27.60	26.21	.....

*Based on records taken at Columbus, 1907–1908.

Table 32.—Seasonal Evaporation in Mad River Valley Above Wright, in Inches

Season	Years					Average
	1915	1916	1917	1918	1919	
Replenishing Period.....	2.38	2.35	1.75	2.43	1.55	2.09
Storage Period.....	3.02	3.05	2.90	3.12	3.65	3.15
Growing Period.....	21:20	18.75	20.05	22.45	22.40	20 97
Total	26.60	24.15	24.70	28.00	27.60	26 21

Table 32 gives the seasonal evaporation for the Mad River Valley calculated from the data in table 31. The data indicates that, on the average, about 80 per cent of the annual evaporation occurs in the growing period, during the months of May to September, inclusive. This leaves only 20 per cent for the replenishing and storage periods, or the seven months from October to April, inclusive.

CHAPTER VI.—RAINFALL AND RUNOFF DURING 1913 FLOOD

The flood of March, 1913, was not only the most severe of which there is record in this valley, but as regards damage was also the greatest that has occurred in the eastern half of the United States since the days of first settlement, or since floods first began to attract attention. A description of this flood and of the damage it wrought has been published in an earlier report.*

It was caused primarily by hard rains which commenced on March 23, and continued with but little interruption until the 27th. Contributing factors were a saturated soil when the rain began, as a result of previous rains, and low temperatures which reduced evaporation to insignificant rates and affected the percolation of water through the soil.

Being the maximum flood on record it was necessary, of course, to make detailed investigations of the rainfall and runoff for use in the design of the flood prevention works. The hydraulic design of the works has been described in volume VII of the technical reports.† The results of such studies were also needed in determining the benefits and damages resulting from the construction of the retarding basins.

This chapter will give the rainfall and runoff date secured, and will discuss the various studies which were made immediately following the flood.

RAINFALL

The daily rainfall over the Miami River drainage area during the storm of March, 1913, is shown in figure 41. Maps are included showing one-inch isohyetals for the 24-hour periods ending at 7 p. m. of March 23, 24, 25, and 26. A map for March 27 has not been reproduced because the precipitation on that day amounted to only about half an inch. Figure 42 shows the total

*The Miami Valley and the 1913 Flood, by Arthur E. Morgan, Chief Engineer, Technical Reports, Part I, The Miami Conservancy District, Dayton, Ohio, 1917.

†Hydraulics of the Miami Flood Control Project, by S. M. Woodward, Technical Reports, Part VII, The Miami Conservancy District, Dayton, Ohio, 1920.

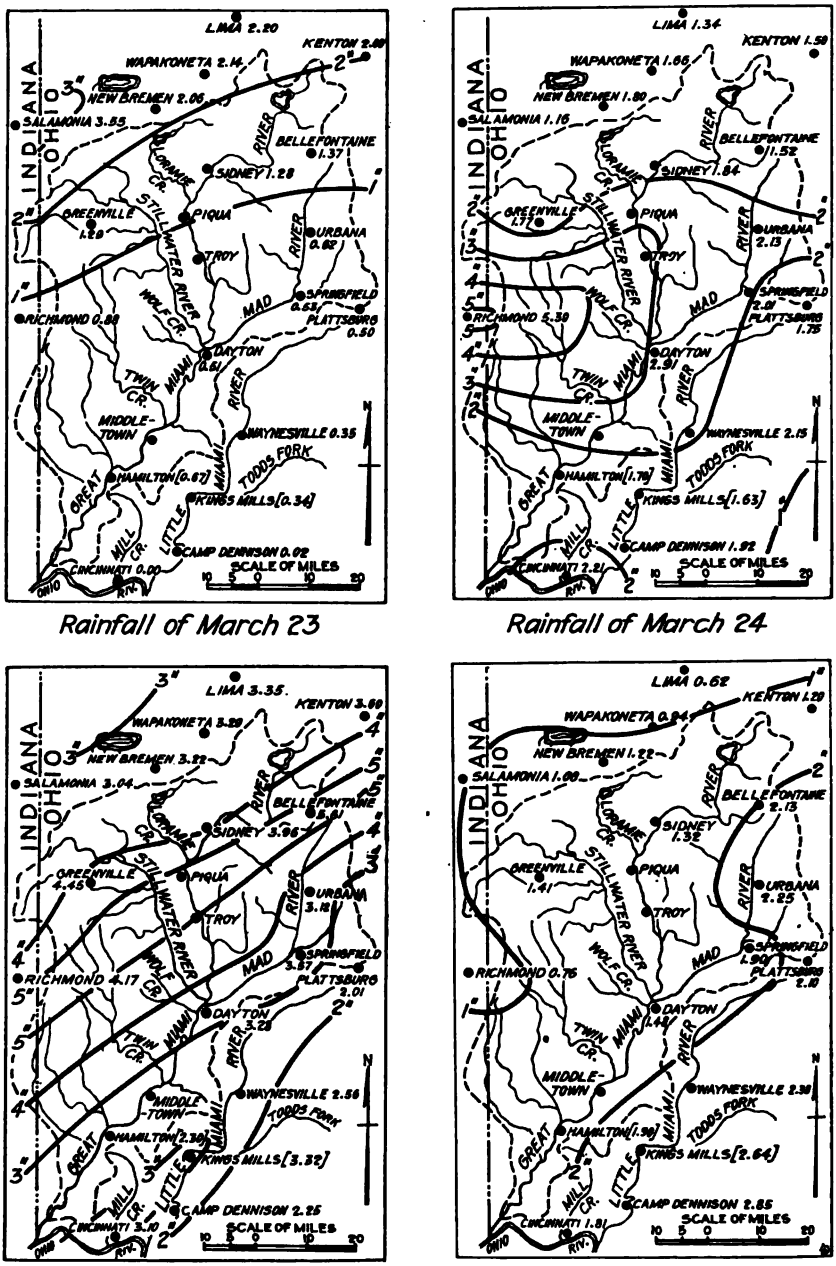
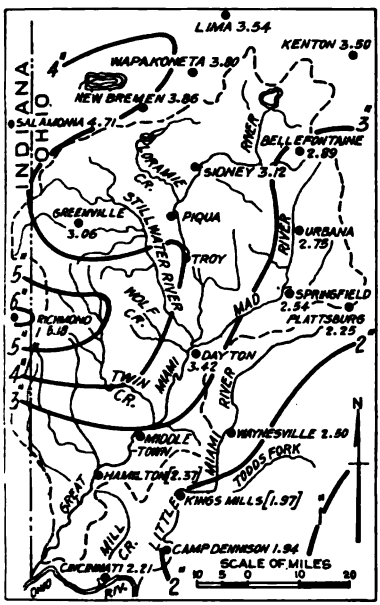
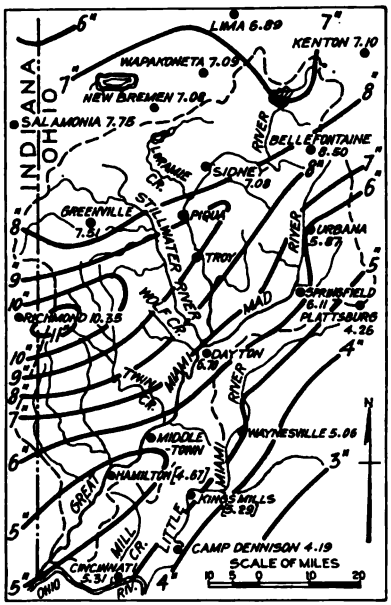


FIG. 41.—DAILY RAINFALL OVER THE MIAMI VALLEY DURING THE STORM OF MARCH, 1913.

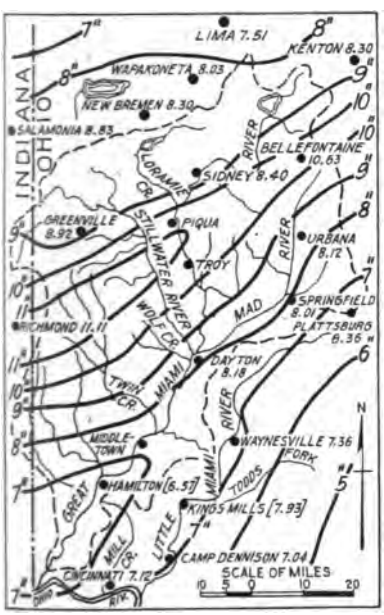
The amounts recorded at the various stations are indicated by the figures written after the names of the stations.



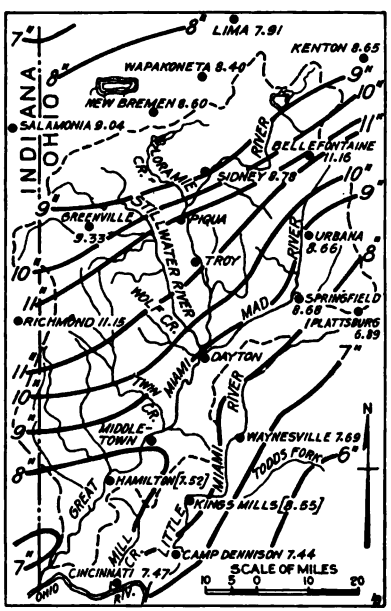
Rainfall, March 23 and 24



Rainfall, March 23 to 25 incl.



Rainfall, March 23 to 26 Incl.



Rainfall, March 23 to 27 incl.

FIG. 42.—CUMULATED RAINFALL OVER THE MIAMI VALLEY DURING THE STORM OF MARCH, 1913.

Rainfall shown on each map is the total from the beginning of the storm up to 7 p. m. of the second date noted under the map.

accumulated precipitation for the periods ending at 7 p. m. of March 24, 25, 26, and 27.

The amounts of rainfall recorded at the various places where gages are maintained are shown by the figures placed after the names of the stations. At the river stations, where the rainfall is measured in the morning, the amounts estimated for the 24-hour periods ending at 7 p. m. are enclosed in brackets.

Fortunately there were a number of well distributed rain gages in this part of the Ohio Valley in 1913. Reports from about 50 stations were utilized in the preparation of figures 41 and 42, many of which were outside the area shown on the maps, some being in Indiana and Kentucky. Although Dayton was the only regular Weather Bureau station located within the Miami Valley at that time, there were several in nearby cities, as at Cincinnati, Indianapolis, Fort Wayne, Toledo, Sandusky, and Columbus. Only a partial graphical record of rainfall was secured at Dayton. Owing to the flooding of the business section of the city, in which the office is located, the triple register could not be kept in operation, the clock stopping at 4:30 p. m. on March 25.

Figure 43 shows the distribution of the precipitation, as regards time, at the above mentioned regular stations, platted from data published by the U. S. Weather Bureau in Bulletin Z.* The abscissas represent time, and the ordinates, the amount of rainfall in inches per hour, the amounts being shown by horizontal lines extending through the hours in which they occurred. Since the actual amounts used in platting the horizontal lines were usually for one-hour periods, the ordinates, in such cases, indicate the total precipitation for each hour as well as the rate. The total rainfall for any length of time at a given station is represented by the area under the portion of the curve corresponding to that time; that is, by the product of the rate and its duration.

Hourly readings were not available for March 27, or for March 26 at Fort Wayne. The precipitation in the latter instance, and probably in the former at some stations, was in the form of snow. Since the precipitation was small in both cases the rates have been computed and platted as having continued

*The Floods of 1913 in the Rivers of the Ohio and Lower Mississippi Valleys, by Alfred J. Henry, Meteorologist, Bulletin Z, U. S. Weather Bureau, Washington, D. C., 1913.

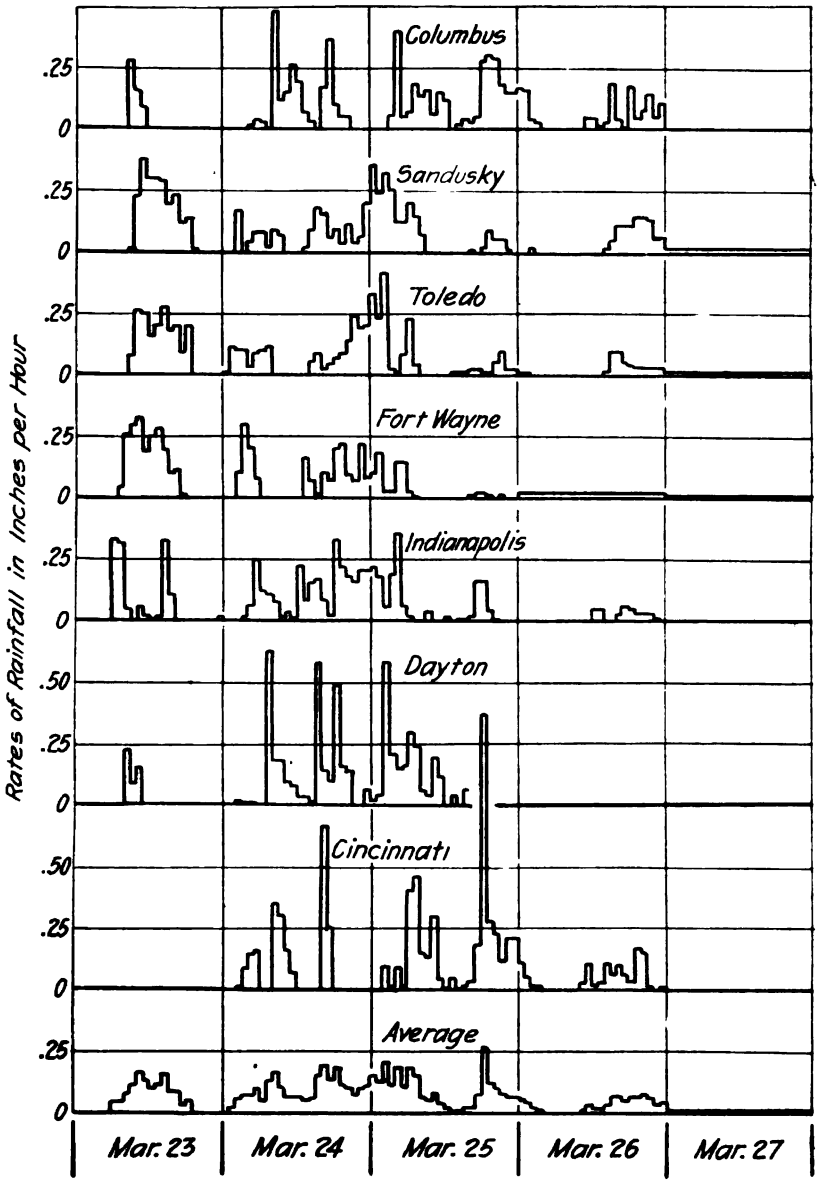


FIG. 43.—HOURLY RAINFALL AT STATIONS NEAR THE MIAMI VALLEY DURING THE STORM OF MARCH, 1913.

Based on graphical records at the U. S. Weather Bureau stations.

uniformly throughout the 24 hours. The actual rainfall on the 27th probably ended by evening or sooner.

It will be noticed that the rain seemed to occur in showers at the various stations, the showers being a little more intense at Cincinnati and Dayton than at the other cities. The times of occurrence of the principal showers at the different places seem to agree fairly well. Consequently, a curve has been added showing the average hourly amount based on the seven actual records. Although the total precipitation over the Miami Valley above Dayton determined from the isohyetal map in figure 42 amounted to 9.60 inches while the total determined from the curve in figure 43 amounted to only 6.40 inches, this curve is the only available indication of the rainfall distribution above Dayton.

It appears that the rainfall began during the morning of the 23rd and continued during the greater part of the day, ending at about the time the daily readings were being taken by the cooperative observers. The average over the Miami Valley above Dayton amounted to 1.20 inches. The precipitation was heaviest in the northern part of the drainage area. The rain began again about midnight and fell almost continuously throughout the 24th, 25th, and 26th. On the 24th the total up to 7 p. m. averaged 2.20 inches above Dayton, the greatest precipitation occurring over the headwaters of Twin Creek, further south than on the preceding day. The greatest rainfall occurred on the 25th, averaging 4.11 inches, and registering a maximum of 5.61 inches at Bellefontaine, about 55 miles northeast of Dayton. It was on the morning of this date that the rivers, which had been steadily rising, overtopped the levees in the principal cities. Near the following midnight the highest stages were attained at places between Dayton and Hamilton. On the 26th the average rainfall amounted to 1.62 inches; on the 27th it amounted to 0.47 inches. The latter was of little consequence, however, as the waters were then everywhere receding rapidly.

RUNOFF

Unfortunately there were only three river gages in the Miami Valley at the time of the flood. These were the gages maintained by the U. S. Weather Bureau at Piqua and Dayton and by the U. S. Geological Survey at Hamilton. Discharge measurements during flood stages had been secured only at Hamilton.

During the summer and fall following the flood, extensive hydrographic surveys were made for the purpose of determining

the maximum rates of discharge at various places along the streams. These surveys and the methods used in calculating the maximum rates of runoff have been described in an earlier report.* The results obtained are shown on the map in figure 44. At each location where measurements were made the quantities are indicated by means of a fraction and a quotient. The numerator represents the maximum total rate of discharge in second feet at the given place; the denominator, the drainage area in square miles above that place; and the quotient, the maximum rate of runoff in second feet per square mile over the given drainage area. It will be noticed that the rates of runoff are unusually high at all places. However, in the case of the smaller drainage areas, these rates continued for but small fractions of a day.

Figure 45 shows the hydrographs of the 1913 flood at Piqua, Dayton, and Hamilton. These were determined as accurately as possible from the gage readings taken during the flood, from the maximum rates of discharge based on the surveys, and from current meter gagings made during subsequent smaller floods. The rates of runoff in inches per day over the drainage areas are platted as ordinates, and the times, as abscissas. For comparative purposes the average curve of hourly rainfall obtained in figure 43 has also been included.

It will be noticed that the rates of runoff above Piqua were lower than those above Dayton and Hamilton, throughout the entire flood period. The rates above Dayton and Hamilton agree very well. It is believed that the reasons for the lower rates of runoff above Piqua are the less intense precipitation above that station, the storage in the Loramie and Lewistown reservoirs, which has been estimated to be equivalent to a depth of about a quarter of an inch over the total drainage area above Piqua, and the somewhat less rolling topography in the upper parts of the valley.

Considering the steeply rising portions of the hydrographs there seems to have been a difference in time of about 6 hours between the Piqua and Dayton curves, and of about 8 hours between the Dayton and Hamilton curves. It will be noticed that at Piqua the river reached its crest at about 10 o'clock Tuesday morning, March 25, and then remained stationary about four hours. At Dayton, however, the maximum stage was not reached until about midnight Tuesday, although the river was within a

*Calculation of Flow in Open Channels, by Ivan E. Houk, Technical Reports, Part IV, The Miami Conservancy District, Dayton, Ohio, 1918.

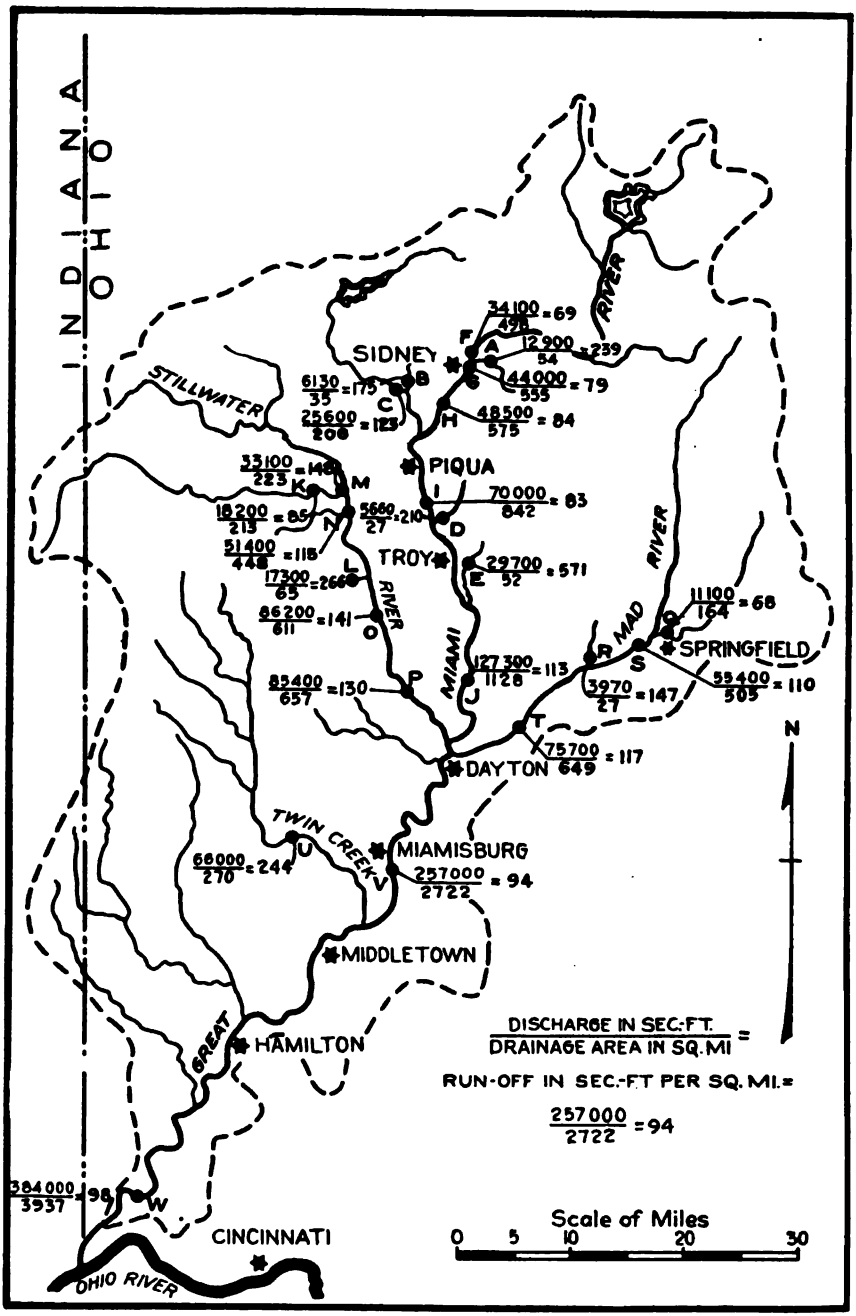


FIG. 44.—MAXIMUM RATES OF RUNOFF IN THE MIAMI VALLEY DURING THE FLOOD OF MARCH, 1913.

The figures are written opposite the places at which measurements were made subsequent to the flood, and give the runoff in second feet per square mile as indicated in the lower right hand corner.

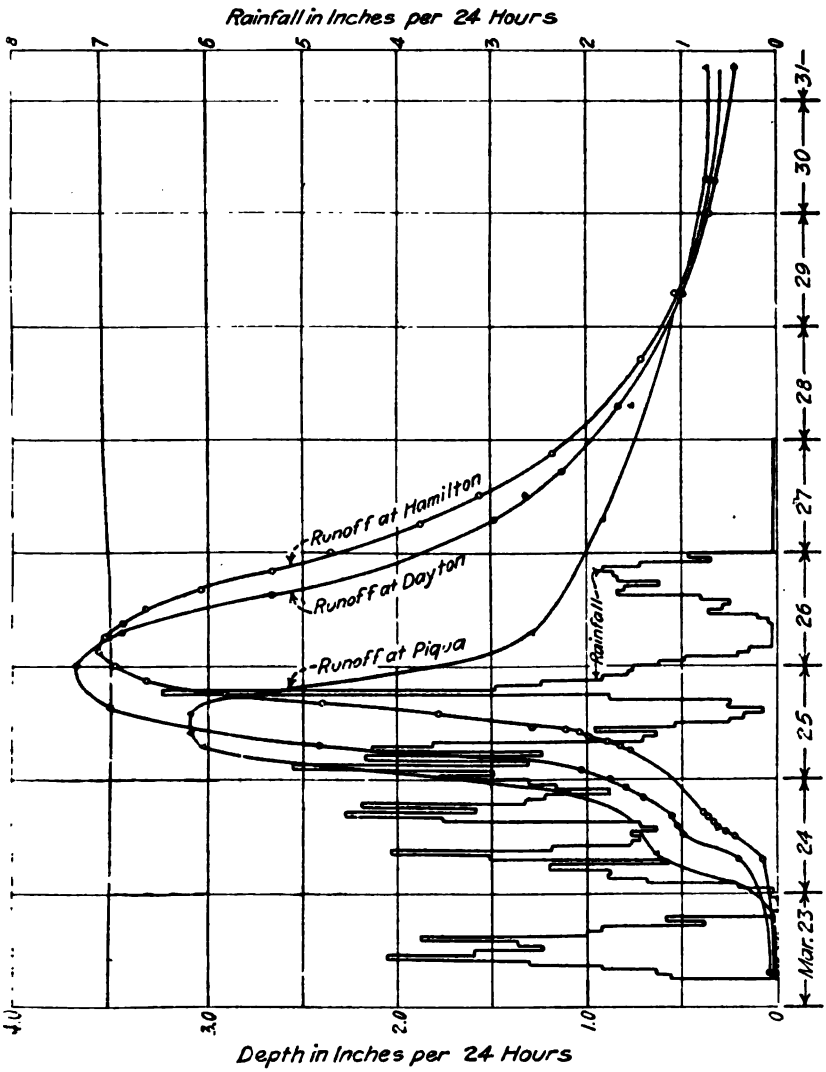


FIG. 45.—HYDROGRAPHS OF 1913 FLOOD AT PIQUA, DAYTON, AND HAMILTON. For comparison the average rainfall curve determined in figure 44 has been added.

foot of its highest stage at noon. At Hamilton the crest was reached at about 3 a. m. Wednesday, March 26. Investigations made within a few weeks after the flood showed that the Stillwater River just above Dayton reached its crest at about noon Tuesday; that the Miami River above its junction with the Stillwater reached its crest at about 7 p. m. Tuesday evening; and that the Mad River above the Miami reached its crest at about midnight Tuesday. In each case the river remained practically stationary for a few hours before it began to fall. Consequently the highest stage at Dayton was due to the coordination of the highest stages in the Upper Miami and Mad Rivers. If the crest in the Stillwater had been delayed a few hours the stage at Dayton would have been higher than it actually was.

Owing to the distribution of the most intense rainfall as regards drainage areas as well as regards time, the highest stages in the various streams were caused by the local runoff rather than by the runoff from the upper drainage areas. No indications of the occurrence of a definite flood wave were found, except, possibly, in the case of Mad River. In this instance the investigations seemed to indicate a difference in the time of crest of about 12 hours between Springfield and Dayton, a distance of about 25 miles. The comparatively slow movement of the crest was due to the great amount of storage in the valley. The Stillwater River was at its crest at practically the same time from Covington to Dayton, a distance of about 30 miles; as was also the Miami from DeGraff to Tippecanoe City a distance of about 45 miles.

RELATION OF RUNOFF TO RAINFALL

Reference to figure 45 shows that while the rainfall curve is comparatively irregular the runoff curves are comparatively smooth. This is because the runoff curves are for large drainage areas, 842 to 3672 square miles. In such cases the irregularities of the rainfall tend to be eliminated by the effects of storage on the ground and in the numerous small tributary drains, as well as by the time required for the runoff to reach the main streams. The conditions may be said to be analagous to the operation of a retarding basin, the rainfall curve corresponding to the inflow to the basin, and the runoff curves, to the outflow. The runoff curve for a drainage area of a few square miles would un-

doubtedly have shown irregularities corresponding to those in the rainfall curve.

It will be noticed that considerable rainfall occurred during the first two days, while in the same period there was comparatively little runoff. It also appears that when the rainfall ceased rather abruptly, the runoff continued for several days afterwards. This condition was due to the surface storage mentioned above, the water held by the small depressions and irregularities in the surface of the ground, as well as in the stream channels, draining out gradually after the rainfall ceased.

In order to consider properly the total surface runoff resulting from the rainfall of March 23 to 27, it is necessary to keep in mind the weather and ground conditions preceding the storm. January was an unusually wet month, the total precipitation, which was well distributed through the month, amounting to over seven inches. February was drier than usual, the rainfall totaling an inch less than the normal of three inches for that month, and occurring mostly in the last three days. March was wet throughout. From the first to the 21st moderate rains were recorded at all of the gaging stations on about ten days. On the 21st the precipitation throughout the valley averaged nearly a half an inch.

It is evident from these conditions that at the beginning of the rain on March 23 the ground was fully saturated, that the ground water flow was greater than usual, and that there was some surface runoff in the streams as a result of the precipitation of March 21. The latter factor can be eliminated in the determination of the flood runoff, caused by the storm of March 23 to 27, by totaling the runoff only to the time when the amount of water in the stream channel was the same as when the flood began. This time was estimated to be in the evening of March 31.

However, the effect of the ground water flow must be allowed for in a different manner, since some runoff was being maintained by underground storage during the entire flood period. From a study of the daily discharges before and after the flood it is estimated that 0.05 of an inch per day should be deducted from the total runoff, in order to obtain the true surface, or flood, runoff. As nearly as can be determined this amount would be the same for Piqua, Dayton, and Hamilton.

Figure 46 shows mass curves of rainfall, flood runoff, and retention for the flood of March, 1913. Rainfall and retention curves are shown only for the total drainage area above Dayton;

flood runoff curves are shown for Piqua, Dayton, and Hamilton. The rainfall curve was calculated from the average curve shown in figure 45, arbitrarily raising the latter so that the total precipitation for each day agreed with the value determined from the isohyetal map in figure 41. The flood runoff curves were likewise calculated from the rate curves of figure 45, deducting the ground water flow from the total, as mentioned above. The retention curve is simply the difference between the rainfall curve and the Dayton flood runoff curve.

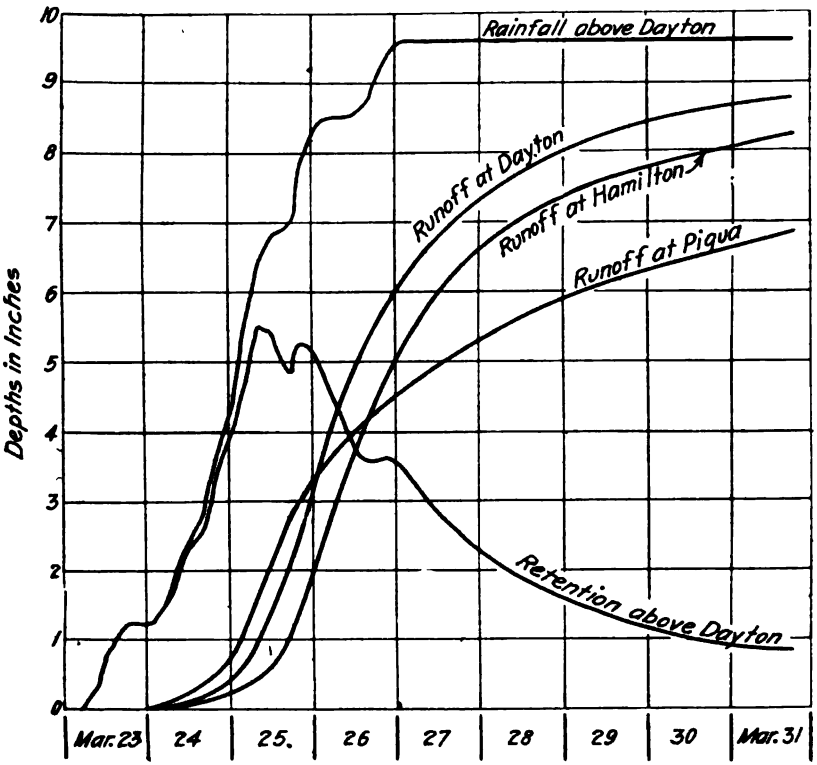


FIG. 46.—RAINFALL, RUNOFF, AND RETENTION DURING THE FLOOD OF MARCH, 1913.

Curves show total values of the various quantities up to any instant.

The retention represents soil absorption, evaporation, and storage on the ground and in the streams. That soil absorption and evaporation were comparatively unimportant during this flood is indicated by the fact that the curve falls rapidly after the most intense precipitation ceased, reaching a value of about 0.84 inches by the evening of March 31. That this would be true

was indicated, of course, by the previous rainfall and weather conditions described above.

The maximum retention occurred at about 8 a. m., March 25, amounting to 5.50 inches. If it is estimated that the total soil absorption and evaporation up to this time had amounted to a half an inch, the storage on the ground and in the streams would be about 5 inches. Of this amount it is probable that the proportion held in the main channels and in the adjacent overflow sections was about 2 inches, and the portion held in the small streams and ponds and on the ground surface, about 3 inches.

Table 33 gives the daily rainfall and flood runoff for each day of the flood, for Piqua, Dayton, and Hamilton. The values

Table 33.—Daily Rainfall and Runoff Above Piqua, Dayton, and Hamilton During the 1913 Flood, in Inches

Date	Piqua		Dayton		Hamilton	
	Rainfall	Runoff	Rainfall	Runoff	Rainfall	Runoff
March 23	1.57	.....	1.20	.....	1.00	.....
24	1.77	0.43	2.20	0.25	2.40	0.10
25	4.14	2.43	4.11	2.12	4.04	1.11
26	1.50	1.41	1.62	3.20	1.62	3.30
27	0.40	0.87	0.47	1.52	0.51	1.85
28	.....	0.64	.....	0.79	.....	0.87
29	.....	0.46	.....	0.46	.....	0.47
30	.....	0.32	.....	0.26	.....	0.29
31	.....	0.29	.....	0.16	.....	0.22
Totals for flood	9.38	6.85	9.60	8.76	9.57	8.21
Ratio of runoff to rainfall	.....	73.0	.....	91.2	.....	85.8
Total retention, in inches	.....	2.53	.....	0.84	.....	1.36

of daily rainfall were obtained from the isohyetal maps in figure 41, and the values of daily runoff, from the mass curves of figure 46. The amounts are for the 24 hours ending at 7 p. m. in all cases. Total quantities for the flood period and ratios of total runoff to total rainfall are given at the bottom of the table.

It will be noticed that the total runoff at Piqua was somewhat smaller than at Dayton and Hamilton. The total at Piqua amounted to 6.85 inches, or to only about 73 per cent of the rainfall, while the total at Dayton was 8.76 inches, or about 91 per cent of the rainfall, and at Hamilton, 8.21 inches, or about 86 per cent of the rainfall. The total amounts at Dayton and Hamilton agree very well. The conditions believed to be responsible for the smaller runoff above Piqua have already been mentioned.

CHAPTER VII.—RAINFALL AND RUNOFF DURING FLOODS SINCE MARCH, 1913

Since March, 1913, only floods of nominal size have occurred in the Miami Valley. During the 1913 flood, the maximum stage at Dayton was 29 feet, or 6 feet above the levees. The highest water since that time was during the flood of April, 1920, when the maximum stage was 16.2 feet, or about 7 feet below the tops of the levees. Practically no flooding has occurred within the cities during this period although farm lands near the various streams have been flooded several times. However, this flooding generally occurred during the winter and early spring months, when but little if any damage was sustained.

RAINFALL, RUNOFF, AND RETENTION DURING FLOODS

Table 34 gives the total rainfall, runoff, and retention in inches, and the ratio of the total runoff to the total rainfall, in per cent, for the larger floods. The dates of the storm periods are given in the table headings. The dates of the flood periods have not been indicated. In general, however, they began shortly after the storm rainfall began and continued from two to five days after the rainfall ceased, the exact time, of course, varying with the nature of the storm and with the topography and extent of the drainage area. For comparative purposes the sizes of the drainage areas above the various stations, have been included. The topography and geology of the Miami Valley have been discussed briefly in chapter I. A map of the valley showing the gaging stations is given in figure 1, page 17.

The values of rainfall included in the table are averages for the drainage areas, for the entire storm periods. They were determined by planimeter measurements on isohyetal maps, similar to those in figures 41 and 42, pages 178 and 179. The values of runoff represent the total flood or surface runoff caused by the indicated rainfall. The runoff maintained by the ground water storage has been deducted in each case, as in the determination of the 1913 flood runoff. The values of retention are simply the

differences between the storm rainfall and the flood runoff. The value for a given storm includes the total amount of moisture taken up by the soil and the total evaporation during the storm period. Storms are arranged chronologically.

The data for the larger drainage areas is believed to be more accurate than most similar data which has been published. The valley is well supplied with rainfall stations, as shown in figure 1; the rating curves on the principal streams have been well developed; and the main portions of the flood hydrographs were determined by special readings taken every hour or every two hours. While the data for the smaller areas is not so accurate, it is believed that with one or two exceptions the errors are not excessive.

DESCRIPTIVE NOTES

On account of the condensed form in which the data in table 34 is presented it has not been feasible to include descriptive notes. Since such notes are desirable in any study of the subject, they are given in the following paragraphs.

July 7-8, 1915

The rainfall in the Miami Valley during the storm of July 7 to 8, 1915, was most intense in the evening of the 7th, from about nine to about ten o'clock. Practically the entire precipitation fell sometime during that evening, only a few hundredths of an inch falling after midnight. Although a half an inch fell on the 11th and 12th, it has not been included in the storm as it caused no surface runoff. The most intense precipitation at Moraine Park has been shown graphically in figure 9, page 80.

The soil moisture experiments carried on at Moraine Park, described in chapter III, show that the soil was not saturated when the rain began, although some rain had fallen each day during the period from July 1 to 5, inclusive, and although the total precipitation during the month of June had been slightly greater than normal. In fact, observations made on the 8th, after the rain had ceased, showed that even then the soil was not saturated to a depth of 6 inches.

The streams began rising about 9 p. m. of the 7th. Highest stages were recorded sometime during the 8th in the upper portions of the valley; on the morning of the 9th at Dayton; and during the evening of the 8th at Hamilton; the crest at Hamilton

Table 34.—Total Rainfall, Runoff, Retention, and Ratio of Runoff to Rainfall during Various Floods since March, 1913

Gaging Station	Drainage Area Square Miles	Storm of July 7–8, 1915				Storm of December 24, 1915, —January 7, 1916				Storm of January 10-13, 1916				Storm of January 26-31, 1916			
		Rainfall in Inches	Runoff in Inches	Retention in Inches	Ratio of Runoff to Rainfall in per cent	Rainfall in Inches	Runoff in Inches	Retention in Inches	Ratio of Runoff to Rainfall in per cent	Rainfall in Inches	Runoff in Inches	Retention in Inches	Ratio of Runoff to Rainfall in per cent	Rainfall in Inches	Runoff in Inches	Retention in Inches	Ratio of Runoff to Rainfall in per cent
Sidney	555	1.94	0.65	1.29	33.	3.90	2.68	1.22	69.	1.56	0.90	0.66	58.	2.56	2.19	0.37	85.
Lockington	255	Gage	install	ed 9–1	3–1915	4.14	3.57	0.57	86.	1.45	1.32	0.13	91.	3.21	2.90	0.31	90
Piqua	842	2.05	1.16	0.89	57.	3.97	3.13	0.84	79.	1.52	1.06	0.46	70.	2.90	2.38	0.52	82.
Tadmor	1128	2.10	0.85	1.25	40.	3.89	2.88	1.01	74.	1.61	0.94	0.67	58.	2.91	2.24	0.67	77.
West Milton	600	2.89	1.40	1.49	48.	3.99	2.99	1.00	75.	1.74	0.81	0.93	47.	3.56	3.40	0.16	95.
Buck Creek	163	1.95	0.21	1.74	11.	3.16	0.60	2.56	19.	1.69	0.59	1.10	35.	1.96	0.62	1.34	32.
Springfield	488	2.09	0.46	1.63	22.	3.65	0.95	2.70	26.	1.83	0.70	1.13	38.	2.07	0.88	1.19	42.
Wright	652	2.09	0.51	1.58	24.	3.41	1.26	2.15	37.	1.82	0.80	1.02	44.	2.33	0.99	1.34	42.
Dayton	2525	2.30	0.81	1.49	35.	3.73	2.47	1.26	66.	1.73	1.03	0.70	60.	2.89	2.13	0.76	74.
Germantown	272	2.99	1.41	1.58	47.	3.30	2.42	0.88	73.	2.09	1.45	0.64	69.	3.47	2.68	0.79	77.
Seven Mile	128	2.53	1.10	1.43	43.	2.68	2.18	0.50	81.	1.85	1.79	0.06	97.	3.27	3.81	—0.54	116.
Four Mile	178	2.16	1.46	0.70	68.	2.82	2.26	0.56	80.	1.93	2.41	—0.48	125.	2.99	4.07	—1.08	136.
Hamilton	3672	2.32	0 75	1.57	32.	3.42	1.55	1.87	45.	1.86	1.14	0.72	61.	2.88	2.09	0.79	73.

Table 34.—*Continued*

Gaging Station	Drainage Area Square Miles	Storm of March 21-22, 1916				Storm of March 25-28, 1916				Storm of May 6-7, 1916				Storm of January 3-6, 1917			
		Rainfall in Inches	Runoff in Inches	Retention in Inches	Ratio of Runoff to Rainfall in per cent	Rainfall in Inches	Runoff in Inches	Retention in Inches	Ratio of Runoff to Rainfall in per cent	Rainfall in Inches	Runoff in Inches	Retention in Inches	Ratio of Runoff to Rainfall in per cent	Rainfall in Inches	Runoff in Inches	Retention in Inches	Ratio of Runoff to Rainfall in per cent
Sidney.....	555	1.48	0.96	0.52	65.	1.98	1.61	0.37	81.	1.85	1.04	0.81	56.	1.81	0.39	1.42	22.
Lockington....	255	1.13	0.99	0.14	88.	1.58	1.25	0.33	79.	2.22	2.00	0.22	90.	2.03	1.01	1.02	50.
Piqua.....	842	1.37	0.97	0.40	71.	1.84	1.54	0.30	84.	2.03	1.37	0.66	67.	1.88	0.71	1.17	38.
Tadmor.....	1128	1.25	0.81	0.44	65.	1.93	1.49	0.44	77.	2.00	1.09	0.91	55.	1.95	0.61	1.34	31.
Pleasant Hill...	453	Gage	install	ed 4-7	-1916	.....	.....	.....	.....	Data	not co	mpute	d	2.14	0.75	1.39	35.
West Milton...	600	0.84	0.49	0.35	58.	1.48	0.93	0.55	63.	1.58	0.75	0.83	47.	2.14	0.67	1.47	31.
Buck Creek....	163	1.04	0.33	0.71	32.	2.20	0.85	1.35	39.	2.58	0.19	2.39	7.	1.72	0.47	1.25	27.
Springfield.....	488	1.19	0.35	0.84	29.	2.34	0.90	1.44	38.	2.52	0.65	1.87	26.	1.90	0.50	1.40	26.
Wright.....	652	1.04	0.45	0.59	43.	2.33	1.03	1.30	44.	2.37	0.60	1.77	25.	1.90	0.44	1.46	23.
Dayton.....	2525	1.04	0.59	0.45	57.	1.89	1.32	0.57	70.	1.97	0.89	1.08	45.	1.99	0.67	1.32	34.
Franklin.....	2785	0.99	0.54	0.45	55.	1.89	1.38	0.51	73.	1.90	0.89	1.01	47.	2.02	0.66	1.36	33.
Germantown...	272	0.43	0	0.43	.....	1.79	0.72	1.07	40.	1.15	0.18	0.97	16.	2.28	1.08	1.20	47.
Seven Mile....	128	0.19	0	0.19	.....	1.79	0.76	1.03	42.	0.70	0.18	0.52	26.	1.90	1.20	0.70	63.
Four Mile.....	178	0.18	0	0.18	.....	1.65	0.55	1.10	33.	0.72	0.17	0.55	24.	2.11	0.82	1.29	39.
Hamilton.....	3672	0.82	0.41	0.41	50.	1.81	1.11	0.70	61.	1.66	0.64	1.02	39.	2.07	0.70	1.37	34.

Table 34.—*Continued*

Gaging Station	Drainage Area Square Miles	Storm of March 11-14, 1917				Storm of June 26-29, 1917				Storm of July 12-17, 1917				Period of December 28, 1917 —February 15, 1918			
		Rainfall in Inches	Runoff in Inches	Retention in Inches	Ratio of Runoff to Rainfall in per cent	Rainfall in Inches	Runoff in Inches	Retention in Inches	Ratio of Runoff to Rainfall in per cent	Rainfall in Inches	Runoff in Inches	Retention in Inches	Ratio of Runoff to Rainfall in per cent	Rainfall in Inches	Runoff in Inches	Retention in Inches	Ratio of Runoff to Rainfall in per cent
Sidney	555	1.74	0.95	0.79	55.	2.27	0.29	1.98	13.	3.72	0.87	2.65	23.	4.02	2.05	1.97	51.
Lockington	255	2.07	1.70	0.37	82.	2.01	0.66	1.35	33.	2.96	1.37	1.59	46.	3.62	2.85	0.77	79.
Piqua	842	1.90	1.30	0.60	68.	2.20	0.47	1.73	21.	3.46	1.23	2.23	36.	3.89	Records missing		
Tadmor	1128	1.92	1.25	0.67	65.	2.38	0.45	1.93	19.	3.07	0.98	2.09	32.	3.95	2.02	1.93	51.
Pleasant Hill	453	2.07	1.67	0.40	81.	2.03	0.84	1.19	41.	2.49	0.92	1.57	37.	4.11	2.60	1.51	63.
West Milton	600	2.09	1.34	0.75	64.	1.86	0.46	1.40	25.	2.31	0.67	1.64	29.	4.04	2.99	1.05	74.
Buck Creek	163	1.73	0.31	1.42	18.	2.17	0.23	1.94	11.	2.37	0.31	2.06	13.	3.87	0.64	3.23	17.
Springfield	488	1.59	0.65	0.94	41.	2.56	0.37	2.19	14.	2.72	0.46	2.26	17.	4.24	0.98	3.26	23.
Wright	652	1.59	0.67	0.92	42.	2.47	0.33	2.14	13.	2.39	0.41	1.98	17.	4.29	1.20	3.09	28.
Dayton	2525	1.90	1.26	0.64	66.	2.25	0.40	1.85	18.	2.67	0.65	2.02	24.	4.07	1.92	2.15	47.
Franklin	2785	1.92	1.30	0.62	68.	2.19	0.45	1.74	21.	2.59	0.65	1.94	25.	4.10	1.57	2.53	38.
Germantown	272	2.63	1.47	1.16	56.	1.42	0	1.42	.....	1.72	0.15	1.57	8.7	4.40	2.32	2.08	53.
Seven Mile	128	1.98	Records Missing			1.61	0	1.61	.....	2.27	0.88	1.39	39.	4.69	3.35	1.34	71.
Four Mile	178	2.40	2.30	0.10	96.	1.56	0	1.56	.....	2.55	0.17	2.38	6.7	4.80	1.63	3.17	34.
Hamilton	3672	1.97	1.24	0.73	63.	2.01	0.34	1.67	17.	2.47	0.58	1.89	23.	4.26	1.38	2.88	32.

Table 34.—*Continued*

Gaging Station	Drainage Area Square Miles	Storm of May 11-13, 1918				Storm of August 25-31, 1918				Storm of March 14-19, 1919				Storm of April 15-21, 1920			
		Rainfall in Inches	Runoff in Inches	Retention in Inches	Ratio of Runoff to Rainfall in per cent	Rainfall in Inches	Runoff in Inches	Retention in Inches	Ratio of Runoff to Rainfall in per cent	Rainfall in Inches	Runoff in Inches	Retention in Inches	Ratio of Runoff to Rainfall in per cent	Rainfall in Inches	Runoff in Inches	Retention in Inches	Ratio of Runoff to Rainfall in per cent
Sidney	555	1.97	0.41	1.56	21.	2.51	0	2.51	.....	3.07	1.72	1.35	56.	5.11	3.88	1.23	76.
Lockington	255	1.64	0.29	1.35	18.	1.87	0	1.87	.....	3.26	2.33	0.93	71.	4.79	4.13	0.66	86.
Piqua	842	1.87	0.38	1.49	20.	2.31	0	2.31	.....	3.14	2.16	0.98	69.	5.00	3.93	1.07	79.
Tadmor	1128	2.03	0.57	1.46	28.	2.69	0.06	2.63	2.2	3.24	1.91	1.33	59.	4.75	3.38	1.37	71.
Pleasant Hill	453	2.06	0.73	1.33	35	2.25	0	2.25	.....	3.48	2.78	0.70	80.	4.50	2.89	1.61	64.
West Milton	600	2.17	0.54	1.63	25.	2.67	0	2.67	.....	3.53	3.04	0.49	86.	4.46	3.08	1.38	69.
Buck Creek	163	3.50	0.48	3.02	14.	4.51	0.54	3.97	12.	2.78	0.75	2.03	27.	5.05	1.13	3.92	22.
Springfield	488	3.27	0.53	2.74	16.	4.17	0.42	3.75	10.	3.10	1.16	1.94	37.	5.12	2.00	3.12	39.
Wright	652	3.29	0.70	2.59	21.	4.30	0.41	3.89	9.5	3.04	1.16	1.88	38.	5.06	1.77	3.29	35.
Dayton	2525	2.45	0.63	1.82	26.	3.18	0.13	3.05	4.1	3.28	2.33	0.95	71.	4.75	2.91	1.84	61.
Franklin	2785	2.53	0.67	1.86	26.	3.19	0.19	3.00	6.0	3.29	2.09	1.20	64.	4.72	2.82	1.90	60.
Germantown	272	2.87	1.23	1.64	43.	3.27	0.42	2.85	13.	4.21	2.50	1.71	59.	4.81	2.46	2.35	51.
Seven Mile	128	3.46	1.89	1.57	55.	3.98	0.74	3.24	19.	4.50	3.04	1.46	68.	4.65	3.57	1.08	77.
Four Mile	178	3.85	2.01	1.84	52.	3.88	0.10	3.78	2.6	4.75	3.40	1.35	72.	4.78	3.61	1.17	75.
Hamilton	3672	2.71	0.80	1.91	30.	3.34	0.18	3.16	5.4	3.48	1.76	1.72	51.	4.70	2.82	1.88	60.

being caused by heavy runoff from the areas below Dayton. The streams then fell rapidly, reached normal stages by the 14th in all cases.

Dec. 24, 1915-Jan. 7, 1916

The flood of the last of December, 1915, and the first of January, 1916, was due to rain falling on a partially melted accumulation of snow. Precipitation occurred nearly every day during the period from December 24 to January 7. During the last of December the temperature at Dayton varied from a few degrees below the freezing point to a few degrees above, so that the precipitation was sometimes snow and sometimes rain. Above Dayton the temperatures were slightly lower. The result was that by the end of the month there was an accumulation of from 6 to 12 inches of snow above Dayton. This was melted by a rainfall of from one to one and a half inches falling on January 1 and 2.

Some flood runoff occurred during the last of December. However, the main part of the flood began on the morning of January 1. The highest stages were reached sometime during the 1st or 2nd, at all places except Hamilton. At this station the crest occurred early in the morning of the 3rd. The surface runoff had drained out completely by the 10th.

The rainfall during November and the first three weeks of December was not greatly different from normal; so that the ground was practically saturated when the storm began. As no cold waves had occurred up to that time the ground could not have contained any appreciable amount of frost.

January 10-13, 1916

The most intense precipitation during the storm of January 10-13, 1916, occurred on the 12th, although appreciable amounts fell on each of the other dates. On the 10th, the first day of the storm, the average rainfall over the different drainage areas varied from 0.15 inches in the Miami Valley above Piqua, to 0.37 inches in the Buck Creek Valley. On the 11th, the amounts were about the same except that they were slightly heavier below Dayton. On the 12th, the amounts varied from 0.51 inches in the Buck Creek Valley to 1.14 inches above Germantown. The following day, the last day of the storm, the rainfall was comparatively light below Dayton. However, above Dayton it varied

from an average of 0.06 inches in the Stillwater Valley to 0.54 inches in the Buck Creek Valley.

The precipitation occurred as rain on the 10th, 11th, and 12th, but changed to snow on the 13th, as the temperature fell with the passing of the storm.

The streams began rising on the 11th and reached their highest stages on the 13th. They then fell rapidly reaching normal stages by the 17th or 18th.

The weather and soil conditions during the last of December, 1915, and the first few days of January, 1916, are indicated by the description of the preceding storm. Comparatively low temperatures from January 6 to 8, inclusive, froze the ground to a depth of from three to five inches, as indicated by observations at Moraine Park. Consequently, the conditions were conducive to high rates of runoff.

January 26-31, 1916

The storm of January 26 to 31, 1916, followed close after the storm of January 10 to 13. The weather from the 13th to the 19th, inclusive, was cold with temperatures as low as 3 degrees below zero at Dayton. Consequently the soil, which had been saturated by the preceding storm, froze to a depth of several inches. The weather warmed up on the 20th, and from a quarter to a half an inch of rain fell each day on the 20th, 21st, and 22nd. From the 23rd to the 26th the weather was warm and fair.

Only a few hundredths of an inch fell on the 26th. On the 27th, 28th, and 29th, the precipitation over the different areas varied from 0.06 to 0.34 inches, from 0.18 to 0.45 inches, and from 0.38 to 0.69 inches, respectively. On the 30th and 31st the most intense precipitation occurred. The amounts varied from 0.67 inches above the Buck Creek station to 1.57 inches above West Milton, on the 30th; and from 0.41 inches above Springfield to 0.94 inches above Lockington, on the 31st.

The rivers began rising on the 28th, reached their highest stages on January 31 and February 1, and then fell rapidly reaching comparatively low stages by February 6 in all cases. The rate of falling was increased somewhat by the comparatively cold weather which followed the storm.

March 21-22, 1916

Practically all of the rainfall during the storm of March 21 to 22, 1916, fell on the 22nd, only a few hundredths of an inch

falling on the 21st. The soil was saturated due to the precipitation of the preceding part of the winter but was not frozen. Although some snow had fallen during the earlier part of the month, it had all melted by the 21st.

The rivers began rising on the 21st, reached their highest stages on the 22nd or 23rd, and then fell rather slowly. The flood runoff has been determined for the period beginning on the 21st and ending on the 26th. Flood runoff after the 26th was not included because the following storm began on the 25th. As there was still some surface runoff in the streams on the 26th, caused by the storm of the 21st and 22nd, the values given in the tables may be slightly low for this storm and slightly high for the following one.

March 25-28, 1916

Only a few hundredths of an inch fell on the 25th. On the 26th the average precipitation for the different drainage areas varied from 0.28 to 1.07 inches, the greater amounts falling in the lower portions of the valley. On the 27th the values varied from 0.35 to 1.81 inches, the greater amounts on this date falling on the Mad River drainage area. The precipitation on the 28th was comparatively small, the averages for the various basins varying from 0.04 to 0.24 inches.

The streams began rising again on the 26th, reached their highest stages on the 27th and 28th, and then fell rapidly. By April 3 the surface runoff had entirely passed the city of Hamilton, the lowest station included in the table.

May 6-7, 1916

The entire precipitation in this storm fell during the night of May 6 and 7. Although a few hundredths of an inch fell during the period from May 10 to 13, no surface runoff resulted and consequently the amounts have not been included in the table.

The ground was fairly wet when the rain began, due to the rains of April and of May 2, 3, and 4, but was not fully saturated. Evaporation and transpiration rates were considerably higher than they had been during the preceding months, but had not yet reached their maximum summer values.

The streams rose rapidly, reaching their crest stages on the 7th and 8th, and then fell rapidly. By the 13th the stage at Hamilton had fallen to a normal value.

January 3-6, 1917

The most intense precipitation during the storm of January 3 to 6, 1917, fell on the 5th and 6th. The rainfall on the 3rd and 4th was comparatively light, amounting to only a few hundredths of an inch each day. On the 5th and 6th the total amounts varied from 1.67 inches in the Miami Valley above Sidney and in the Buck Creek Valley above Springfield to 2.12 inches in the Twin Creek Valley above Germantown.

The soil was practically saturated when the rain began and was not frozen, although some freezing weather had occurred during the preceding month. There was no snow on the ground. The precipitation during the entire storm was in the form of rain.

The rivers began rising on the 5th, reached their highest stages on the 5th and 6th, and then fell steadily. The total flood runoff had passed Hamilton by the 10th.

March 11-14, 1917

The heaviest rainfall during the storm of March 11 to 14, 1917, fell on the 13th, the amounts on that day varying from 0.95 inches above the Wright, Seven Mile, and Four Mile stations, to 1.49 inches above Pleasant Hill. The rainfall averaged about a half an inch throughout the valley on the 11th, and from 0.05 to 0.37 inches on the 12th, the heaviest precipitation on the latter date falling in the southern part of the valley. On the 14th the rainfall varied from 0.07 inches in the Mad River Valley to 0.38 inches in the Seven Mile basin.

The ground was saturated when the rain began due to the precipitation of the preceding winter. The rainfall was greater than normal during January. Although the precipitation was less than normal during February the meteorological conditions were not such as to dry the ground to any appreciable depth. Some rain fell in the valley on each day of March preceding the storm except the 1st, 9th, and 10th.

The rivers began rising on the 11th, fell slightly on the 12th, and then began rising again on the 13th, reaching crest stages on the 13th and 14th. The flood runoff had passed Hamilton by the 20th.

Light rainfall occurred on the 16th, 17th, and 18th, but no surface runoff was caused and consequently the amounts have not been included in the table.

June 26-29, 1917

Only a few hundredths of an inch fell on June 26, the first day of the storm. On the 27th, the average precipitation on the different drainage areas varied from 0.48 inches above Lockington to 1.08 inches above Wright. On the 28th, the amounts varied from 0.63 inches above Germantown to 1.13 inches above Springfield. On the 29th, the rainfall varied from 0.03 inches in the Buck Creek Valley to 0.56 inches above Sidney and Tadmor.

The ground was in ordinary June condition, being neither unusually dry nor unusually wet. Although considerable precipitation had occurred during the preceding part of the month it had been utilized by the comparatively high evaporation and transpiration rates which occur during this part of the year.

The streams began rising on the 28th, reached their highest stages on the 28th and 29th, and then fell slowly. The total surface runoff had passed Hamilton by the 5th.

Although some precipitation occurred on July 2, no appreciable surface runoff resulted. Consequently the amounts have not been included in the table.

July 12-17, 1917

The storm of July 12 to 17, 1917, followed close after the preceding described storm. The rainfall on July 12 varied from 0.22 inches above Germantown to 0.88 inches above Sidney. On the 13th the amounts varied from 0.17 inches above Sidney to 0.47 inches above Pleasant Hill. On the 14th, the day of heaviest precipitation, the amounts varied from 0.65 inches in the Four Mile Creek Valley to 1.72 inches above Sidney. On the 15th the precipitation was comparatively light, the amounts being less than a quarter of an inch except in the upper Miami Valley where they varied from 0.29 inches above Tadmor to 0.39 inches above Lockington. The rainfall was light throughout the valley on the 16th. On the 17th, the last day of the storm, the amounts varied from 0.11 inches above Lockington to 1.17 inches in the Four Mile Creek basin. The ground was fairly wet when the rain began, due to previous rainfall, but was not saturated.

The rivers began rising on the 13th, reached their maximum stages on the 14th and 15th, and then fell slowly, The total flood runoff had passed Hamilton by the 22nd.

Dec. 28, 1917, to Feb. 15, 1918

The flood of February, 1918, was caused almost entirely by melting snow. The winter of 1917 and 1918 was noted for its

severity. The occurrence of heavy snows and severe cold waves began early in December. The greater part of the precipitation between December 3 and February 11 occurred as snow, the temperature being below freezing the greater part of the time, sometimes several degrees below zero.

The ground froze to a depth of a few inches during the cold period from December 6 to 18, but thawed out and became saturated during the warmer period from the 19th to the 27th. The snows of the early part of December melted during the latter period, the melting being hastened somewhat by light rainfall occurring on the 23rd, 24th, and 25th. The ground then froze again, due to colder weather, and remained frozen until the thawing period of February.

During the period from December 28 to January 3, inclusive, the snowfall throughout the valley was equivalent to about 0.65 inches of rain. This snow was partially melted by a rainfall of about an inch occurring during the period from the 5th to the 8th. Freezing weather began again, however, before much surface runoff could occur. From January 11 to February 6 the snowfall was equivalent to about 2 inches of rain. The actual amounts varied from 1.60 inches above Lockington to 2.58 inches above the Four Mile Creek station. From February 7 to 15 the precipitation amounted to about a half an inch throughout the valley. This was in the form of rain and was distributed over several days time.

Small rises occurred in January. However, the principal flood runoff began February 9. The streams rose rather irregularly reaching their highest stages on the 12th or 13th. Ice jams occurred in many places. The flood runoff had drained out by the 18th.

May 11-13, 1918

The most intense precipitation during the storm of May 11 to 13, 1918, fell on the 12th and 13th, less than a half an inch falling on the 11th. The soil was in ordinary May condition when the rain began. It was not saturated although the precipitation during the preceding month had been greater than normal.

The streams began rising on the 12th, and reached their highest stages on the 13th. They then fell rather uniformly, reaching normal stages by the 18th or 19th.

August 25-31, 1918

The precipitation during the storm of August 25 to 31 was heaviest in the Mad River Valley although heavy rainfall occurred throughout the Miami River drainage area. Only a few hundredths of an inch fell on the 25th. However, the rest of the precipitation was distributed rather uniformly throughout the remainder of the storm period.

The soil was neither unusually dry nor unusually wet when the rain began. The streams in the Mad River Valley began rising on the 30th and reached crest stages on the 30th or 31st. But little, if any, flood runoff occurred in the other portions of the valley. By September 3 the total surface runoff had passed Hamilton.

March 14-19, 1919

Although the precipitation during February and the first part of March had been less than normal, the ground was practically saturated when the storm of March 14 to 19, 1919, began. The precipitation on the 14th was comparatively light, amounting to only a few hundredths of an inch. The heaviest rainfall of the storm occurred on the 15th. The total precipitation on the 14th and 15th, up to 7 p.m., of the latter date, varied from 1.15 inches above Sidney to 2.47 inches above the Four Mile Creek station. On the following day the rainfall was not quite so intense, the amounts varying from 0.77 inches in the Buck Creek Valley to 1.54 inches in the Twin Creek Valley. On the 17th and 18th the rainfall was nearly uniform throughout the Miami Valley, the total amounting to about 0.75 inches on the 17th, and to about 0.25 inches on the 18th. Only a few hundreds of an inch fell on the 19th. The intensities at Moraine Park on the 15th, 16th, and 17th, are shown in figure 10, page 81.

The rivers began rising on the 15th, reached their crest stages on the 16th and 17th, and then fell uniformly, reaching normal stages in all parts of the valley by the 24th.

April 15-21, 1920

The storm of April 15 to 21, 1920, caused the highest stage that has occurred at Dayton since the great flood of March, 1913. Only a few hundredths of an inch fell on the 15th. On the 16th the precipitation was considerably heavier. The total amounts for the 15th and 16th varied from 0.83 inches in the

Seven Mile Valley to 1.92 inches above Sidney. Averages varying from 0.25 to 0.50 inches per day over the valley fell on the 17th, 18th, and 19th. On the 20th and 21st the precipitation was again heavy, the daily amounts varying from about a half an inch to about 2 inches in the different parts of the valley. The rainfall was slightly greater on the 20th than on the 21st except in the Mad River Valley.

The ground was nearly saturated when the rain began, due to rains of March and the early part of April.

The rivers began rising on the 16th, fell on the 17th, 18th, and 19th, and then rose rapidly on the 20th, reaching crest stages on the 20th and 21st. They then fell rapidly, reaching normal stages throughout the valley by the 28th.

TOTAL RETENTION

A study of the data in table 34 shows that during similar storms the total retention is generally greater in the Mad River drainage area than in the other portions of the Miami Valley. For instance, during the storm of December 24, 1915, to January 7, 1916, the total retention above Springfield, resulting from a precipitation of 3.65 inches, amounted to 2.70 inches; while in the Stillwater Valley above West Milton the retention, due to a precipitation of 3.99 inches, was only 1.00 inch. In the Miami Valley above Tadmor the retention caused by a precipitation of 3.89 inches amounted to 1.01 inches, or practically the same as in the Stillwater Valley. Other storms show similar conditions, although the differences are not always so great. The total retention in the Buck Creek Valley is generally a little greater than in the other parts of the Mad River drainage area.

As explained in chapter V, the relatively high retention in the Mad River Valley is due to the comparatively loose and shallow surface soil, underlain by extensive deposits of gravel.

The total retention during the summer storms seems to be much greater than during similar winter storms, as would, of course, be expected, due to the relatively higher rates of evaporation and soil absorption and the greater amounts of available surface and soil storage during the summer. This is well shown by a comparison of the storms of August 25 to 31, 1918, and April 15 to 21, 1920. These storms were very similar as regards duration and intensity. The total precipitation during the August storm was somewhat smaller than during the April storm, especially in the Stillwater and Upper Miami Valleys.

However, in spite of the smaller rainfall, the retention during the August storm was greater than during the September storm in every catchment area, the differences varying from 0.50 inches above Germantown to 2.61 inches above the Four Mile Creek station in all instances except Buck Creek. In this case the difference was only 0.05 inches.

The negative values of total retention obtained for the Seven Mile and Four Mile Creek valleys for the storms of January 10 to 13 and 26 to 31, 1916, are probably in error. This may be due to either or both of two causes. The runoff may be too large due to the difficulties encountered in securing accurate records on a flashy stream of this nature; or the rainfall may be too small due to the occurrence of heavy showers between rain gaging stations. It is believed that the latter reason accounts for the greater parts of the discrepancies. For similar reasons some of the data for the smaller drainage areas for other storms may be in error.

As before mentioned the total retention for a given storm period represents the total quantity of water taken up by the soil plus the total quantity evaporated. Rates of soil absorption and evaporation both vary widely due to variations in meteorological, topographical, and geological conditions. Consequently it does not seem feasible to estimate the value of each component during the various storms given in table 34.

Soil absorption and evaporation are both relatively high in the summer and relatively low in the winter. The former is probably greater if the rainfall is steady, than it is if the rain falls in separate intense showers. The latter is greater when the rain falls intermittently, especially if the showers are separated by intervals of warm windy weather. In the case of forested areas appreciable amounts of the precipitation are intercepted by the foliage of the trees and are evaporated after the rain ceases without ever having reached the ground.

While the areas covered by forests in the Miami Valley are relatively small some data on interception has been secured. During the summer and fall of 1919 two rain gages were maintained by the writer at his residence. One was located in the open and the other under a hackberry tree, about forty feet high, about midway between the trunk and the outer edge of the foliage. The data secured is given in full in table 35.

It will be noticed that the quantities intercepted vary considerably. During slow steady rains from 0.10 to 0.15 inches are

Table 35.- Rainfall Intercepted by Hackberry Trees at Dayton

Date	Rain		Precipation in Open in Inches	Intercepted by Trees in Inches	Remarks
	Began	Ended			
7-31-19	4 a.m.	6 a.m.	0.43	0.12	Steady rain.
7-31-19	2 p.m.	2:15 p.m.	0.47	0	High wind.
8- 1-19	d.n.	d.n.	0.03	0.03	Steady rain.
8- 5-19	p.m.	6 p.m.	0.07	0.07	Steady rain.
8- 6-19	d.n.	d.n.	0.12	0.03	Steady rain.
8-13-19	p.m.	d.n.	0.25	0.14	Light showers.
8-16-19	11:25 a.m.	11:40 a.m.	0.80	0	Strong wind, some hail.
8-24-19	5 p.m.	6 p.m.	0.15	0.10	Steady rain.
8-24-19	7 p.m.	9 p.m.	0.95	0.02	Thunder-showers.
8-31-19	5 p.m.	8 p.m.	0.21	0.10	Showers.
9-19-19	d.n.	8 a.m.	0.14	0.07	Showers with some wind.
9-21-19	d.n.	d.n.	0.77	0.17	Showers with some wind.
9-21-19	3 p.m.	6 p.m.	0.42	0.15	Showers, clear and windy during forenoon.
*10- 4-19	5 p.m.	d.n.	0.49	0.26	Light showers, windy between showers.
10- 5-19	noon	d.n.	1.02	0.20	Heavy showers, some wind.
10- 9-19	d.n.	7:30 a.m.	0.35	0.12	Steady rain, no wind.
10- 9-19	7:30 a.m.	9 a.m.	0.13	0.02	Steady rain, no wind.
10-10-19	10 a.m.	d.n.	0.83	0.43	Showers, clear between.
10-14-19	d.n.	7:30 a.m.	0.16	0.07	Steady rain.
10-15-19	d.n.	7:30 a.m.	0.25	0.07	Light showers.
10-21-19	d.n.	noon	0.21	0.11	Showers.
10-25-19	d.n.	7:30 a.m.	0.06	0.04	Light showers.
10-26-19	d.n.	5 p.m.	0.62	0.26	Showers.
10-27-19	d.n.	6:30 a.m.	0.43	0.22	Showers.
10-29-19	noon	6 p.m.	0.27	0.14	Showers.
10-31-19	5:30 a.m.	7 a.m.	0.20	0.12	Showers.
10-31-19	7 a.m.	6 p.m.	0.92	0.24	Showers.
11- 7-19	d.n.	7 a.m.	0.51	0.11	Steady rain.
11-10-19	8 a.m.	6 p.m.	0.42	0.11	Showers.
5-23-20	6 p.m.	6:30 p.m.	0.62	0.16	Thunder-showers, some wind.
6- 2-20	6:35 p.m.	7:10 p.m.	1.25	0.15	Strong wind for about 10 minutes, some hail.
6- 9-20	8 p.m.	9 p.m.	0.34	0.19	Steady rain.
6-14-20	d.n.	d.n.	0.38	0.23	Showers, warm and windy between showers.

intercepted. If the precipitation occurs as showers, separated by clear, windy intervals, the quantities intercepted are considerably higher, due to the evaporation between showers.

MAXIMUM VALUES OF RETENTION

Table 36 gives the maximum values of retention, in inches, during a few of the larger storms. The quantities were obtained from mass curves of rainfall and runoff similar to those shown in figure 46. They are not as definitely determined as they would have been if data on hourly rainfall had been available. Except in a few instances the rainfall data was limited to daily amounts. Had more exact information been at hand the values of maximum retention would probably have been increased slightly.

Table 36.—Maximum Retention, in Inches, During Various Floods

Gaging Station	Dates of Storms					
	July 7–8 1915	January 10–13 1916	January 26–31 1916	March 21–22 1916	March 25–28 1916	March 14–19 1919
Sidney	1.94	1.20	1.41	1.15	1.17	2.13
Lockington	.....	1.03	1.30	0.71	0.60	1.63
Piqua	2.04	1.12	1.58	1.07	1.13	1.91
Tadmor	2.08	1.30	1.91	1.13	1.31	2.30
Pleasant Hill	.....	.....	.....	.....	.....	1.81
West Milton	2.88	1.50	1.89	0.65	1.01	1.59
Buck Creek	1.95	1.15	1.42	0.79	1.51	2.50
Springfield	2.02	1.25	1.38	0.92	1.64	2.24
Wright	2.09	1.28	1.66	0.87	1.80	2.29
Dayton	2.28	1.36	1.93	0.94	1.46	2.26
Franklin	.....	.....	.....	0.96	1.47	2.33
Germantown	2.96	1.38	1.51	*	1.36	2.19
Seven Mile	2.53	0.97	0.64	*	1.24	2.55
Four Mile	2.16	0.59	0.96	*	1.34	2.49
Hamilton	2.32	1.40	1.87	0.78	1.43	2.45

*No flood runoff.

The maximum retention occurred sometime prior to the occurrence of the crest stages. In the case of the July, 1915, storm they occurred about the time the rain ceased. Since practically the entire rainfall fell in about an hour in this instance, the maximum values of retention are nearly as great as the total precipitation. Other conditions being the same the values tend to be lower on the smaller drainage areas due to the comparatively short time required for the runoff to reach the gaging sections.

The maximum value of retention, being the maximum difference between the mass curves of rainfall and runoff, repre-

sents the accumulated soil absorption plus the accumulated evaporation plus the actual quantity of water held on the ground or in the streams at the time the maximum retention occurs. The accumulated soil absorption plus the accumulated evaporation cannot be definitely determined except for the entire flood period. However, it is possible, by a proper consideration of the duration of the storm and of the time of occurrence of the maximum retention, to estimate the proportion of the total absorption and evaporation which has occurred up to the time of the maximum retention. These estimated quantities may then be deducted from the values of maximum retention, thus obtaining estimated values of the maximum quantities stored on the ground surface and in the streams.

Table 37 gives the estimated values of accumulated soil absorption and evaporation at the time of the maximum retention, and also the resulting estimates of maximum surface storage, for the storms included in table 36.

The volumes of water stored in the channels of the streams may be directly calculated whenever data is available regarding water levels, area of cross sections, and distances. Table 38 gives the maximum volumes stored in certain lengths of the principal streams of the Miami Valley during the floods of July, 1917, March, 1919, and March, 1913, calculated in this manner.

The flood of July, 1917, was a comparatively small freshet, not exceeding the channel capacity at any place in the valley. The maximum rates of discharge were only from 5 to 8 per cent of the maximum rates which occurred during the 1913 flood. The flood of March, 1919, was somewhat larger. It caused some overflow in practically all parts of the valley outside the cities. The maximum rates of discharge in this case were from 10 to 20 per cent of the maximum 1913 rates.

The volumes given for the March, 1919, flood do not include the storage on the overflowed lands outside of the main channels. However, the volumes given for the March, 1913, flood do include such storage. In the latter instance the volumes on the overflowed areas were much larger than the volumes within the channels themselves.

Reference to table 38 shows that the quantities held in the main channels during the floods of July, 1917, and March, 1919, were relatively small. Since these quantities represent the storage in the channels when the crest stages occurred, the quantities in the channels when the maximum surface storage took place

Table 37.—Soil Absorption, Evaporation, and Maximum Surface Storage during Various Floods

Gaging Stations	Total Soil Absorption and Evaporation up to Time of Maximum Retention — Floods of						Maximum Surface Storage — Floods of					
	July 7-8, 1915	January 10-13, 1916	January 26-31, 1916	March 21-22, 1916	March 25-28, 1916	March 14-19, 1919	July 7-8, 1915	January 10-13, 1916	January 26-31, 1916	March 21-22, 1916	March 25-28, 1916	March 14-19, 1919
Sidney	1.29	0.50	0.30	0.52	0.25	1.01	0.65	0.70	1.11	0.63	0.92	1.12
Lockington	.....	0.10	0.25	0.14	0.22	0.70	.....	0.93	1.05	0.57	0.38	0.93
Piqua	0.89	0.35	0.42	0.40	0.20	0.73	1.15	0.77	1.16	0.67	0.93	1.18
Tadmor	1.25	0.50	0.54	0.44	0.29	1.00	0.83	0.80	1.37	0.69	1.02	1.30
Pleasant Hill	.....	.....	.....	.....	.....	0.52	.....	.....	.....	.....	.....	1.29
West Milton	1.49	0.70	0.13	0.35	0.37	0.37	1.39	0.80	1.76	0.30	0.64	1.22
Buck Creek	1.74	0.83	1.07	0.71	0.90	1.52	0.21	0.32	0.35	0.08	0.61	0.98
Springfield	1.63	0.85	0.95	0.84	0.96	1.46	0.39	0.40	0.43	0.08	0.68	0.78
Wright	1.58	0.77	1.07	0.59	0.87	1.41	0.51	0.51	0.59	0.28	0.93	0.88
Dayton	1.49	0.53	0.61	0.45	0.38	0.71	0.79	0.83	1.32	0.49	1.08	1.55
Franklin	.....	.....	.....	0.45	0.34	0.90	.....	.....	.....	0.51	1.13	1.43
Germantown	1.58	0.48	0.63	0.43	0.80	1.28	1.38	0.90	0.88	.....	0.56	0.91
Seven Mile	1.43	0.04	.....	0.19	0.77	1.10	1.10	0.93	.....	.....	0.47	1.45
Four Mile	0.70	.....	.....	0.18	0.83	1.01	1.46	.....	.....	.....	0.51	1.48
Hamilton	1.57	0.54	0.63	0.41	0.47	1.29	0.75	0.86	1.24	0.37	0.96	1.16

must have been still smaller. A study of the data in table 37 and 38 shows that the greater parts of the maximum surface storage during the floods included in table 37 must have been on the ground, in the small pools and depressions, and in the channels of the smaller tributary streams, rather than in the main channels.

RATES OF RAINFALL AND RUNOFF

Table 39 gives the maximum daily rates of rainfall and runoff in inches over the drainage areas, and the absolute maximum rate of runoff, in inches per 24 hours, for the greater number of the storms included in table 34. The greatest 24-hour runoff is not the maximum amount calculated for the 24 hours ending at 7 p.m., but is the amount for the 24 hours in which the greatest runoff occurred. It was obtained graphi-

Table 38.—Storage in River Channels During Various Floods

River	Length	Drainage Area* Square Miles	Storage in Inches Flood of		
			July 1917	March 1919	March 1913
Stillwater.....	Pleasant Hill to Dayton..	674	0.16	0.33	1.48
Miami.....	Loramie Creek to Dayton.	1162	0.17	0.21	1.64
Mad.....	Springfield to Dayton....	689	0.07	0.09	1.79
Miami.....	Dayton to Hamilton.....	3672	0.14	0.22	1.19
Total of 3 lengths above Dayton.....		2525	0.14	0.21	1.64
Total of 4 lengths above Hamilton.....		3672	0.24	0.36	2 31

*Total above lower end of length.

cally after the hydrographs had been platted. However, the maximum 24-hour rainfall is the maximum for the 24 hours ending at 7 a.m. or 7 p.m., generally the latter, rather than the actual maximum. The available rainfall data was not sufficient to determine the true 24-hour maximum. The ratio of the greatest daily runoff to the greatest daily rainfall, in per cent, is also included.

It will be noticed that no very unusual 24-hour rainfalls have ocurred during the period covered by the table. With the exception of the summer storms of July, 1915, and May, 1916, the quantities seldom amount to as much as two inches. The runoff during the summer storms was relatively less than during the winter storms as already mentioned. Consequently the

Table 39.—Rates of Rainfall and Runoff During Various Storms

Gaging Station	Drainage Area Square Miles	Storm of July 7–8, 1915				Storm of January 10–13, 1916				Storm of January 26–31, 1916				Storm of March 21–22, 1916			
		Maximum 24-hour Rainfall in Inches	Maximum 24-hour Runoff in Inches	Ratio of Maximum Runoff to Maximum Rainfall in per cent	Maximum Rate of Runoff in Inches per 24 Hours	Maximum 24-hour Rainfall in Inches	Maximum 24-hour Runoff in Inches	Ratio of Maximum Runoff to Maximum Rainfall in per cent	Maximum Rate of Runoff in Inches per 24 Hours	Maximum 24-hour Rainfall in Inches	Maximum 24-hour Runoff in Inches	Ratio of Maximum Runoff to Maximum Rainfall in per cent	Maximum Rate of Runoff in Inches per 24 Hours	Maximum 24-hour Rainfall in Inches	Maximum 24-hour Runoff in Inches	Ratio of Maximum Runoff to Maximum Rainfall in per cent	Maximum Rate of Runoff in Inches per 24 Hours
Sidney	555	1.94	0.27	14.	0.32	1.08	0.34	31.	0.38	1.08	0.76	70.	0.82	1.48	0.38	26.	0.40
Lockington	255	Gage	install	ed 9–1	3–1916	0.98	0.39	40.	0.47	1.37	1.18	86.	1.46	1.13	0.35	31.	0.57
Piqua	842	2.05	0.47	23.	0.58	1.04	0.38	37.	0.43	1.18	0.79	67.	0.91	1.37	0.35	26.	0.42
Tadmor	1128	2.10	0.37	18.	0.46	1.00	0.35	35.	0.39	1.14	0.79	69.	0.89	1.25	0.28	22.	0.31
Pleasant Hill	453	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
West Milton	600	2.89	0.63	22.	0.69	1.02	0.33	32.	0.39	1.57	1.06	68.	1.49	0.84	0.22	26.	0.23
Buck Creek	163	1.95	0.14	7.	0.16	0.83	0.32	39.	0.34	0.67	0.32	48.	0.36	1.04	0.25	24.	0.32
Springfield	488	2.04	0.32	16	0.37	0.91	0.40	44.	0.47	0.83	0.33	40.	0.40	1.19	0.27	23.	0.33
Wright	652	2.09	0.26	12.	0.31	0.91	0.37	41.	0.43	0.78	0.31	40.	0.34	1.04	0.23	22.	0.26
Dayton	2525	2.30	0.37	16.	0.41	0.99	0.40	40.	0.45	1.13	0.69	61.	0.75	1.04	0.25	24.	0.27
Franklin	2785	.....	.....	.....	.....	.....	.....	.....	.....	Gage	install	ed 3–1	5–1916	0.99	0.23	23.	0.25
Germantown	272	2.99	1.00	33.	1.61	1.14	0.75	66.	0.98	1.38	0.95	69.	1.23	0.43	No F	lood R	unoff
Seven Mile	128	2.53	0.78	31.	1.80	0.97	0.93	96.	1.37	1.20	1.42	118.	2.41	0.19	No F	lood R	unoff
Four Mile	178	2.16	0.54	25.	0.75	0.84	1.10	131.	1.84	1.22	1.36	111.	1.84	0.18	No F	lood R	unoff
Hamilton	3672	2.32	0.28	12.	0.29	1.04	0.39	37.	0.43	1.08	0.55	51.	0.57	0.82	0.16	20.	0.17

Table 39.—*Continued*

Gaging Station	Drainage Area Square Miles	Storm of March 25–28, 1916				Storm of May 6–7, 1916				Storm of January 3–6, 1917				Storm of March 11–14, 1917			
		Maximum 24-hour Rainfall in Inches	Maximum 24-hour Runoff in Inches	Ratio of Maximum Runoff to Maximum Rainfall in per cent	Maximum Rate of Runoff in Inches per 24 Hours	Maximum 24-hour Rainfall in Inches	Maximum 24-hour Runoff in Inches	Ratio of Maximum Runoff to Maximum Rainfall in per cent	Maximum Rate of Runoff in Inches per 24 Hours	Maximum 24-hour Rainfall in Inches	Maximum 24-hour Runoff in Inches	Ratio of Maximum Runoff to Maximum Rainfall in per cent	Maximum Rate of Runoff in Inches per 24 Hours	Maximum 24-hour Rainfall in Inches	Maximum 24-hour Runoff in Inches	Ratio of Maximum Runoff to Maximum Rainfall in per cent	Maximum Rate of Runoff in Inches per 24 Hours
Sidney	555	1.50	0.60	40.	0.65	1.91	0.57	30.	0.72	1.67	0.17	10.	0.23	1.14	0.34	30.	0.37
Lockington	255	1.01	0.58	57.	0.85	2.24	1.24	55.	2.06	1.92	0.50	26.	0.71	1.36	0.67	49.	0.95
Piqua	842	1.34	0.54	40.	0.58	2.03	0.71	35.	1.01	1.75	0.29	17.	0.35	1.22	0.44	36.	0.51
Tadmor	1128	1.39	0.50	36.	0.55	2.00	0.52	26.	0.67	1.82	0.26	14.	0.32	1.20	0.41	34.	0.50
Pleasant Hill	453	Gage	install	ed 4–7	-1916	1.53	0.50	33.	0.57	2.01	0.34	17.	0.36	1.49	0.56	38.	0.59
West Milton	600	1.00	0.40	40.	0.43	1.58	0.40	25.	0.48	2.02	0.32	16.	0.34	1.42	0.52	37.	0.53
Buck Creek	163	1.75	0.41	23.	0.68	2.59	Small	Rise	Only	1.67	0.30	18.	0.41	1.00	0.23	23.	0.36
Springfield	488	1.79	0.49	27.	0.61	2.52	0.42	17.	0.57	1.80	0.27	15.	0.33	1 02	0.30	29.	0.37
Wright	652	1.81	0.46	25.	0.51	2.37	0.35	15.	0.43	1.81	0.25	14.	0.28	0.95	0.28	29.	0.31
Dayton	2525	1.32	0.47	36.	0.52	1.97	0.43	22.	0.51	1.87	0.29	16.	0.32	1.20	0.41	34.	0 44
Franklin	2785	1.26	0.47	37.	0.50	1.90	0.41	22.	0.46	1.88	0.26	14.	0.28	1.19	0.39	33.	0 41
Germantown	272	1.00	0.44	44.	0.51	1 15	0.17	15.	0.29	2.12	0.61	29.	0.75	1.56	0.71	46.	0.96
Seven Mile	128	1.07	0.41	38.	0 47	0.72	0.14	19.	0.26	1.69	0.73	43.	0.90	0.95	Recor	ds mis	sing
Four Mile	178	1.05	0.29	28.	0.33	0.71	0.10	14.	0.15	1.84	0.52	28.	0.82	.....	0.73	.....	2.93
Hamilton	3672	1.06	0.33	31.	0.34	1.66	0.24	14.	0.25	1.92	0.27	14.	0.31	1.17	0.35	30.	0.37

Table 39.—*Continued*

Gaging Station	Drainage Area Square Miles	Storm of June 26–29, 1917				Storm of July 12–17, 1917				Storm of March 14–19, 1919				Storm of April 15–21, 1920			
		Maximum 24-hour Rainfall in Inches	Maximum 24-hour Runoff in Inches	Ratio of Maximum Runoff to Maximum Rainfall in per cent	Maximum Rate of Runoff in Inches per 24 Hours	Maximum 24-hour Rainfall in Inches	Maximum 24-hour Runoff in Inches	Ratio of Maximum Runoff to Maximum Rainfall in per cent	Maximum Rate of Runoff in Inches per 24 Hours	Maximum 24-hour Rainfall in Inches	Maximum 24-hour Runoff in Inches	Ratio of Maximum Runoff to Maximum Rainfall in per cent	Maximum Rate of Runoff in Inches per 24 Hours	Maximum 24-hour Rainfall in Inches	Maximum 24-hour Runoff in Inches	Ratio of Maximum Runoff to Maximum Rainfall in per cent	Maximum Rate of Runoff in Inches per 24 Hours
Sidney	555	1.15	0.13	11.	0.17	1.72	0.30	17.	0.36	1.15	0.45	39.	0.48	1.17	0.82	70.	0.90
Lockington	255	1.10	0.31	28.	0.36	1.60	0.47	29.	0.66	1.33	0.73	55.	0.90	1.08	1.20	111.	1.72
Piqua	842	1.16	0.21	18.	0.24	1.70	0.40	24.	0.50	1.21	0.56	46.	0.61	1.01	0.82	81.	0.91
Tadmor	1128	1.30	0.20	15.	0.23	1.57	0.28	18.	0.31	1.25	0.44	35.	0.47	1.08	0.90	83.	0.99
Pleasant Hill	453	0.98	0.34	35.	0.45	1.40	0.44	31.	0.50	1.58	0.76	48.	0.81	1.40	0.85	61.	0.89
West Milton	600	0.88	0.26	30.	0.34	1.28	0.35	27.	0.37	1.59	0.94	59.	1.00	1.44	0.97	68.	1.02
Buck Creek	163	1.08	No F	lood R	unoff	1.25	0.21	17.	0.23	1.24	0.23	19.	0.32	1.55	0.78	50.	0.96
Springfield	488	1.27	0.15	12.	0.18	1.66	0.27	16.	0.34	1.23	0.34	28.	0.40	1.52	0.77	51.	0.89
Wright	652	1.20	0.15	12.	0.18	1.38	0.18	13.	0.20	1.21	0.36	30.	0.39	1.50	0.71	48.	0.95
Dayton	2525	1.14	0.18	16.	0.19	1.41	0.25	18.	0.27	1.34	0.60	45.	0.64	1.27	0.83	65.	0.87
Franklin	2785	1.10	0.17	15.	0.19	1.37	0.24	18.	0.26	1.37	0.52	38.	0.54	1.33	0.67	51.	0.69
Germantown	272	0.63	No F	lood R	unoff	0.90	0.10	11.	0.14	1.65	0.91	55.	1.16	1.91	1.18	62.	1.38
Seven Mile	128	0.88	No F	lood R	unoff	.....	0.49	.....	1.28	2.12	1.16	55.	1.54	1.91	1.98	104.	3.96
Four Mile	178	0.95	No F	lood R	unoff	0.65	Data n	ot com	puted	2.47	1.40	57.	1.86	1.93	1.72	89.	2.37
Hamilton	3672	0.98	0.11	11.	0.12	1.22	0.17	14.	0.18	1.48	0.49	33.	0.50	1.46	0.59	41.	0.60

Table 40.—Ratio of Maximum 24 Hour Discharge to Absolute Maximum Discharge, in Per Cent

Gaging Stations	Drainage Area Square Miles	DATES OF STORMS														
		July 7-8, 1915	December 24, 1915 Jan. 7, 1916	January 10-13, 1916	January 26-31, 1916	March 21-22, 1916	March 25-28, 1916	May 6-7, 1916	Jan. uary 3-6, 1917	March 11-14, 1917	June 26-29, 1917	July 12-17, 1917	May 11-13, 1918	August 25-31, 1918	March 14-19, 1919	April 15-21, 1920
Sidney	555	85	92	89	93	95	93	79	76	93	80	81	87	*	93	91
Lockington	255	.....	76	83	81	62	69	60	69	72	84	71	92	*	81	69
Piqua	842	80	87	90	86	84	93	70	81	86	87	79	90	*	91	90
Tadmor	1128	79	87	89	88	89	89	78	81	83	86	91	96	83	96	91
Pleasant Hill	453	.....	.....	.....	.....	.....	.....	87	93	94	76	89	97	*	95	96
West Milton	600	92	90	84	71	97	91	83	93	98	76	93	95	*	94	96
Buck Creek	163	86	71	93	87	78	60	*	72	63	*	90	68	50	71	81
Springfield	488	88	81	87	80	81	80	73	84	81	83	80	72	74	85	87
Wright	652	84	93	85	92	87	91	81	90	91	81	88	88	92	93	75
Dayton	2525	90	93	90	92	92	91	84	90	93	94	92	92	85	96	95
Franklin	2785	.....	.....	.....	.....	92	93	88	92	95	91	91	94	89	97	97
Germantown	272	62	72	76	78	*	86	57	82	74	*	70	72	67	79	85
Seven Mile	128	44	88	68	59	*	87	56	81	**	*	39	52	90	75	50
Four Mile	178	72	95	60	74	*	88	71	64	25	*	*	56	*	75	72
Hamilton	3672	96	98	90	95	97	97	96	86	92	91	94	93	91	97	97

*No appreciable rise. **Records not obtained.

values of the 24-hour runoff are not unusually large. On the larger drainage areas they do not materially exceed an inch. On the smaller areas they seldom amount to more than an inch and a half. The ratios of the 24-hour runoff to the 24-hour rainfall, of course, vary widely, owing to the different characteristics of the storms as well as to the different soil and surface conditions.

The maximum rates of runoff in inches per day are, of course, somewhat larger than the average rates for the 24-hour periods. On the larger areas they sometimes amount to about an inch and a half. On the smaller areas they occasionally amount to from three to four inches.

Table 40 gives the ratios, in per cent, of the maximum 24-hour rates of discharge to the absolute maximum rates, for various floods. In order to conform with the arrangement in table 34, the storm dates have been given rather than the flood dates. Data for the flood of February, 1918, which was caused by melting snow, has not been included since the hydrographs were very erratic due to ice jams.

The values for a given station vary considerably, as would be expected, due principally to the different durations of the most intense precipitation. For the storms where practically the entire precipitation fell in one day, such as those of July 7 to 8, 1915, and May 6 to 7, 1916, the values are generally smaller than where the heavy rainfall continued through two or more days, as in the storms of January 26 to 31, 1916, and March 14 to 19, 1919. The values are, of course, generally smaller for the smaller drainage areas.

CHAPTER VIII.—FLOOD FORECASTING

One of the outgrowths of the rainfall and runoff investigations described in the preceding chapters has been the development of the flood forecasting system now maintained by the Miami Conservancy District. Steps toward the inauguration of this system were taken during the summer of 1916, following the freshets of January, February, March, and May of that year. It was recognized then that such a system would be a necessity during the coming construction period, and that the preliminary steps should be taken at once in order that by the time construction began a thoroughly established information service might be available to the contractors as one of the assets of the job.

Although the United States Weather Bureau was maintaining a flood warning service in the Miami Valley at that time, which it had inaugurated several years before, their forecasts were limited to rises in which flood stages were reached or exceeded. Such a service was, of course, wholly inadequate for the purposes of the District. Inasmuch as it would be necessary to locate the construction equipment to a considerable extent in the valleys along the river channels, or actually within the channels, where sudden rises of only two or three feet might cause considerable damage, it was felt that an independent system should be developed by which the necessary forecasts could be made and issued without the interposition of a third party at critical times. The necessity of adopting such a course became still more evident when it was considered that in several instances the construction work would be located at places where only four to six hours elapse between the occurrence of the most intense rainfall and the maximum stages.

It should be stated here that the most hearty cooperation has always existed between the officials of the Weather Bureau and of the Conservancy District and that this cooperation has resulted in a considerable saving in expense to both parties. Moreover, the agreement and accuracy of the forecasts issued from the two sources during critical times has resulted in a feeling of

confidence and security on the part of the people throughout the entire valley.

THE PRESENT SERVICE

At the present time forecasts of crest stages and the times of their occurrence are being made for the five dams and for all of the principal cities along the Miami River below Sidney. In the case of the dams and of the cities where construction work is being carried on in the river channels, forecasts are made for rises of all magnitude, even as small as one or two feet. For the other places forecasts are made only when the rises are great enough to cause some damage or to cause apprehension on the part of the public. However, complete information regarding river conditions is always available at the headquarters office for all places, even though the magnitude of the rise is negligible.

Warnings are issued to the construction engineers or contractors at the various places, or to other interested parties. During critical times bulletins regarding river conditions are issued to the public through the local newspapers or by posting in conspicuous places. Information is also widely distributed by telephone. Operators are kept on duty during the entire night so as to furnish desired information at any time. During one freshet two operators were kept busy answering such inquiries from early morning until midnight. These inquiries come not only from the residents of Dayton but also from residents of practically all parts of the valley, from Sidney on the north to the Ohio River on the south.

During long continued storms it is necessary to issue several forecasts. Final estimates of maximum stages to be attained cannot be made until the most intense precipitation has occurred and the weather conditions are such that no additional heavy rainfall is expected. At such times the preliminary forecasts are based on the total rainfall occurring up to the time the forecasts are made. These are issued with the information that the rain is expected to continue and that the forecasted stages will be exceeded by amounts depending on the amount of the subsequent rainfall. For weather forecasts the officials of the District rely entirely o nthe work of the U. S. Weather Bureau.

REPORTS BEING SECURED

In order to maintain the above described service it is, of course, necessary to receive numerous reports from all parts of the valley. Cooperative arrangements were made with the U. S. Weather Bureau in June, 1916, by which special rainfall and river reports from their observers are secured at the headquarters office of the District during storm periods, as well as at the local office of the Weather Bureau. Special reports at such times are also secured from the greater number of the stations maintained by the Miami Conservancy District. These reports are made by telephone or telegraph whenever 0.70 of an inch of rain falls in 24 hours or less, or whenever there is a sudden rise of three or more feet in the river stage. A confirmation of each report is made by mail as soon as the message has been telegraphed or telephoned.

The gaging stations in the Miami Valley are shown in figure 1, page 17. Reports are received from all of the combined river and rainfall stations, from all but one or two of the rainfall stations, and from nearly all of the river stations located above Hamilton. While it may appear to the reader at first that reports from so many stations are not necessary, it must be remembered that reports from all stations at the proper time cannot be expected. Observers have other work to do besides attending to their gages. Although the importance of making the reports should be emphasized to the observers, and although the members of the observers' families should be trained to take readings and send reports, it frequently happens that for some legitimate reason the report is not made. Such occurrences must be expected in a flood forecasting system and must be allowed for by arranging for more reports than are absolutely necessary. Too many reports do no harm while too few result in poor predictions.

Although the observers are permitted to send these reports by either telegraph or telephone, they are instructed to send them by telephone whenever it is feasible to do so. The advantages of receiving the messages by telephone are, first, that the forecaster can question the observers regarding existing rainfall, river, or weather conditions, thus obtaining desired information which they might otherwise neglect to furnish even though instructed previously to do so; and, second, that the observers, in reporting by telephone, are necessarily kept on the job until the message is delivered, thus insuring more prompt

delivery. Quite frequently it has happened that reports handed to telegraph operators in small towns have been delayed so long in transmission that they arrived too late to be of use.

One factor which has been of great assistance in the forecasting work of the District and which has made possible the forecasting of small rises of one or two feet in the principal streams, is the direct telephone communication maintained between the headquarters office and the division offices at the five dams. Private wires are maintained to each dam. Consequently it is posible to obtain accurate information from trained observers, directly interested in the work, at any time, without the trouble involved in putting through long distance calls by the usual methods.

The river observers are instructed to report the latest gage reading and time of observation; the time the rise began and the stage at that time; whether the river is rising, stationary, or falling; the maximum stage and time of occurrence, if the river is falling; and such general information regarding rainfall, snowfall, runoff, and weather as they may possess. Rainfall observers are instructed to report the time of beginning of rain; the time of reading the gage and the amount of the precipitation up to that time; whether or not it is still raining; the time of ending of the rain, if it has ended; the amount and character of the precipitation during the preceding week; and such general information regarding the nature of the rainfall, snowfall, runoff, and weather, as they may have observed.

FORECASTING METHODS

Owing to the comparatively short intervals of time required for the flood runoff to reach the main streams, flood forecasting in the Miami Valley is of necessity largely based on the forecaster's judgment and on his familiarity with previous floods. With reports coming in from all parts of the valley and with people clamoring for information simultaneously, there is no opportunity to sit down and quietly analyze the problem or to apply theoretical formulas.

The forecaster keeps in touch with the weather and soil conditions throughout the valley at all times so that when the rain begins he knows approximately how much precipitation will be required to fill the surface storage and what proportion of the remainder may be expected to be taken up by the soil. This information in conjuction with the rainfall reports and his knowl-

edge of similar previous floods enables him to forecast the maximum stages and the times at which they will be reached at the principal stations in the upper portions of the valley as soon as the most intense precipitation has ceased. Preliminary forecasts are, of course, made as the reports arrive, as previously noted, but maximum stages can not be forecasted until the most

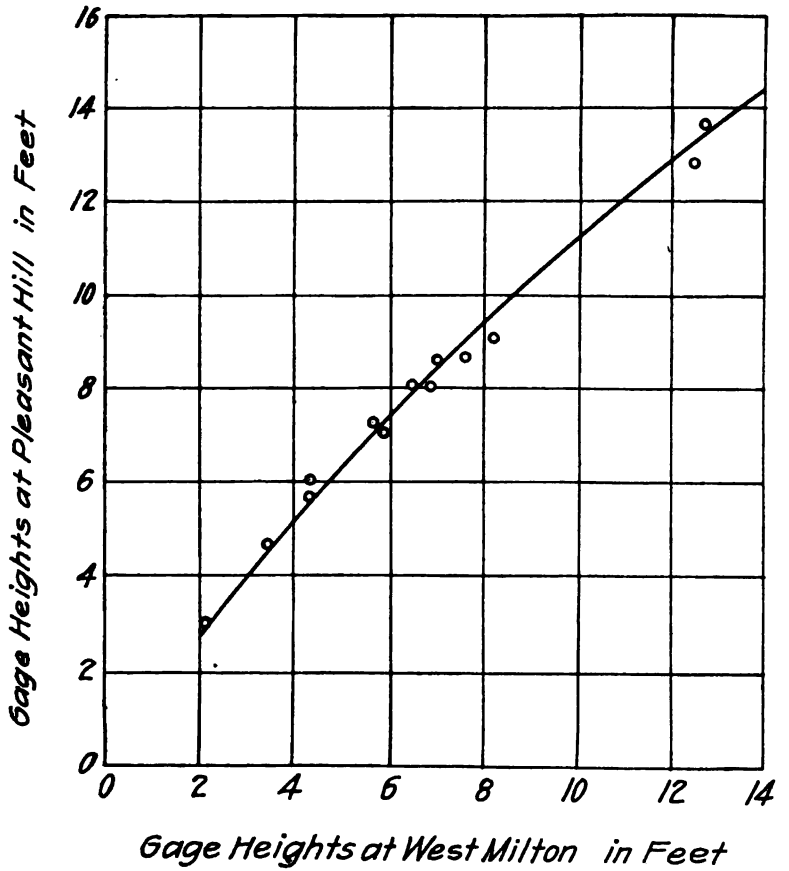


FIG. 47.—RELATION BETWEEN CREST STAGES ON THE STILLWATER RIVER AT PLEASANT HILL AND WEST MILTON.

intense precipitation has occurred. Having estimated the maximum stages to be attained in the upper portions of the valley, crest stages at the lower stations on the main streams above Dayton are forecasted by the aid of crest relation diagrams, proper consideration being given to channel storage and rainfall intensities below the upper stations.

Figure 47 shows the relation between crest stages at the

Pleasant Hill and West Milton stations on the Stillwater River; figure 48 shows a similar relation for the Springfield and Wright stations on Mad River; and figure 49 shows similar relations for the Sidney, Piqua, and Tadmor stations on the upper Miami River. Points vary from the curves somewhat in all instances, due to local variations in rainfall intensity. This is especially noticeable in the case of the Piqua-Tadmor curve, given in figure 49. Points falling to the left of the curve represent storms in which the most intense precipitation occurred

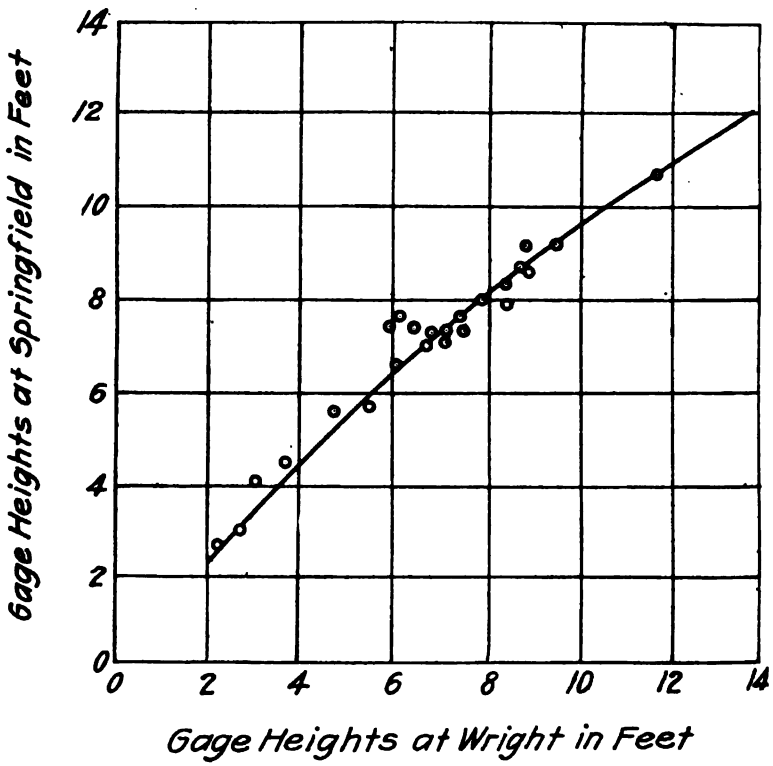


FIG. 48.—RELATION BETWEEN CREST STAGES ON THE MAD RIVER AT SPRINGFIELD AND WRIGHT.

over the northwestern portion of the drainage area; while points falling to the right represent storms where the heavy rain fell mostly on the eastern portions of the valley. Discrepancies due to this cause are less noticeable on the Sidney-Tadmor curve because heavy rainfall falling east of Piqua drains northward toward Sidney as well as southward toward Tadmor, thus producing similar conditions at both places. It is for this reason

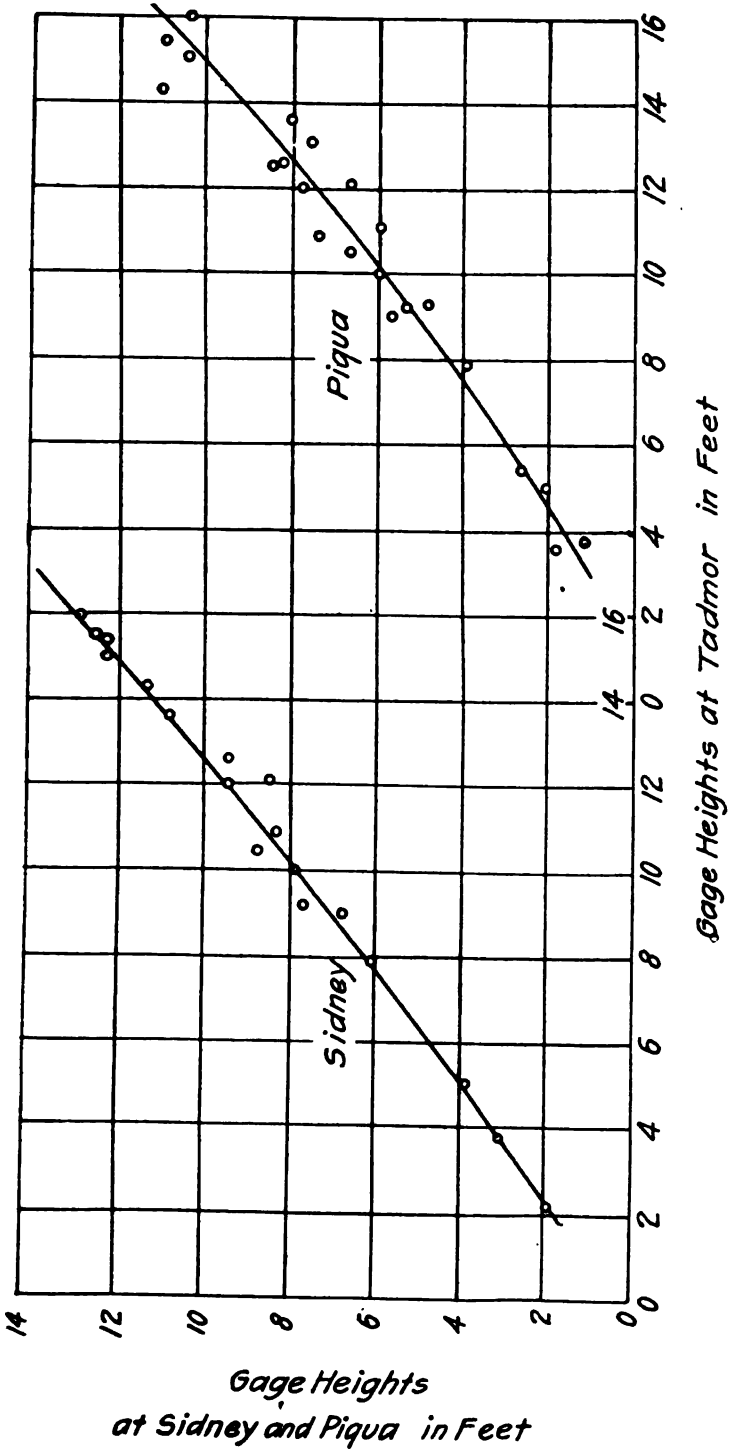


FIG. 49.—RELATIONS BETWEEN CREST STAGES ON THE MIAMI RIVER AT SIDNEY, PIQUA, AND TADMOR.

that the crest stage frequently occurs at Tadmor at the same time as at Sidney.

Forecasts of stages at Dayton are based on the estimated crests at the upper stations, proper allowance being made for lack of coordination of crests from the three streams. Wolf Creek is so steep, so short, and its drainage area so small, in comparison with the Stillwater, Mad, and Miami Rivers, that its flood runoff always passes Dayton several hours before the crests from the other three streams arrive. Only in very exceptional cases does any appreciable amount of flood runoff from the Wolf Creek valley reach Dayton at the time the Dayton crest is attained.

The maximum discharges from the Stillwater, Miami, and Mad Rivers do not reach Dayton in any definite order, although the crest discharge from the upper Miami practically always arrives later than that from one of the other two streams. The exact order of arrival varies with the rainfall intensity, location of most intense precipitation, and time of occurrence on the different areas. During the storm of July 7 to 8, 1915, in which practically all of the rain fell between the hours of 9 and 10 p.m. of the 7th, at all stations in the valley, the crest discharge from the Stillwater River reached Dayton about 24 hours after the rain had ceased, that from the Mad River arrived in about 16 hours, and that from the upper Miami, in about 29 hours. During the storm of May 6 to 7, 1916, in which practically all of the precipitation occurred in the night of the 6th and 7th, the crest discharge from the Stillwater River arrived at Dayton about 23 hours after the rain ceased, that from the Mad River arrived in about 25 hours, and that from the Miami, in about 29 hours. The difference in the time of arrival at Dayton of the crest discharge from Mad River during these two floods is found in the location of the most intense precipitation. During the May storm the heaviest rainfall in the Mad River Valley occurred near Urbana, about 40 miles by river from Dayton, while during the July storm it fell near Springfield, only about 25 miles from Dayton.

Having estimated the maximum stage to be reached at Dayton, forecasts for Miamisburg and Franklin may be made by the aid of the crest relation diagrams shown in figure 50. Similar methods may be used for Middletown and Hamilton whenever the rainfall distribution is such that the crest stages at these places are caused by the flood runoff from the drainage areas

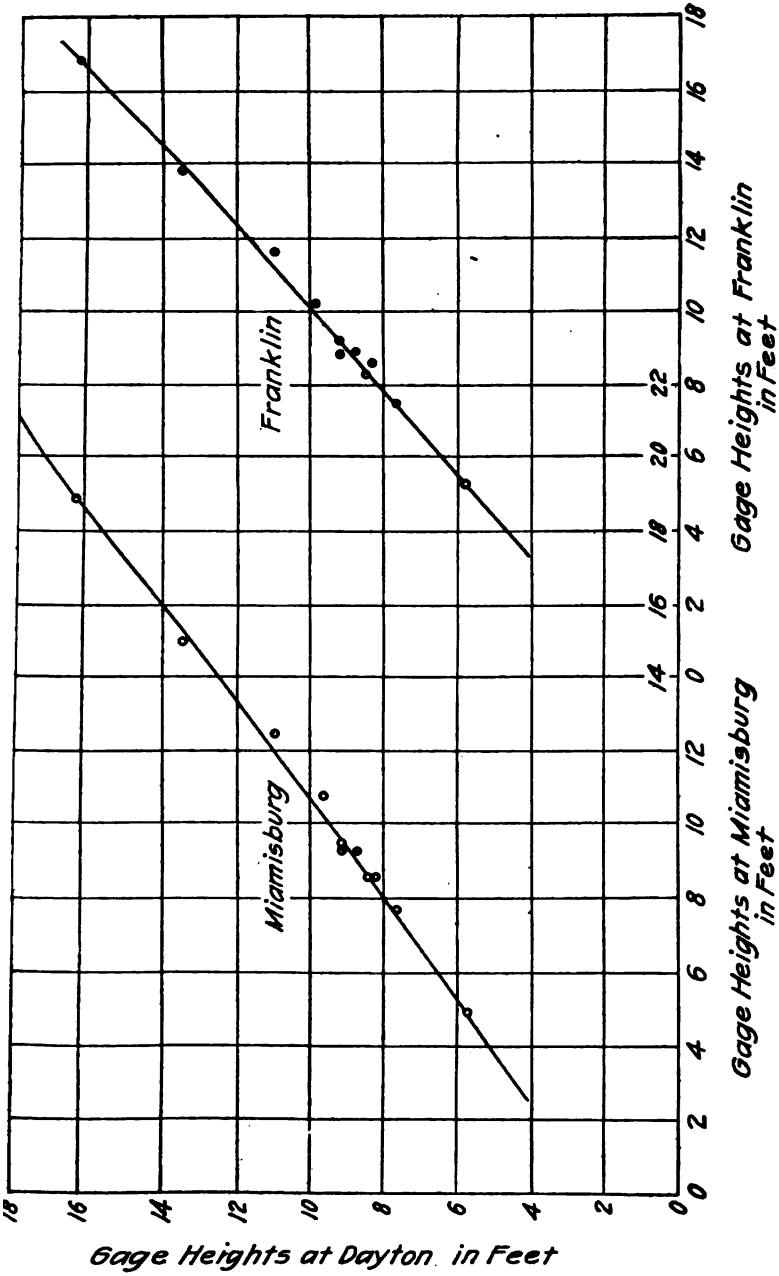


FIG. 50.—RELATIONS BETWEEN CREST STAGES ON THE MIAMI RIVER AT DAYTON, MIAMISBURG, AND FRANKLIN.

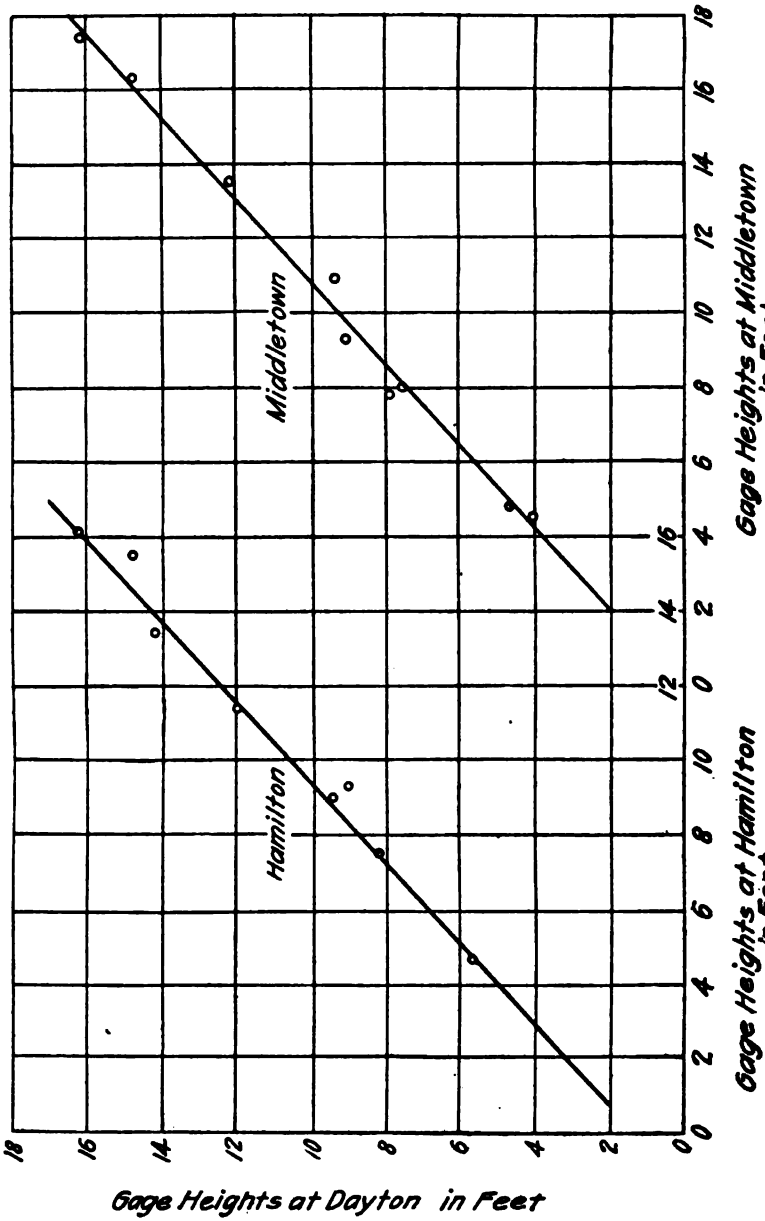


FIG. 51.—RELATIONS BETWEEN CREST STAGES ON THE MIAMI RIVER AT DAYTON, MIDDLETOWN, AND HAMILTON.

above Dayton. Diagrams for use in such instances are shown in figure 51. It frequently happens, however, that the crest stages at points below the mouth of Twin Creek are caused by the flood runoff from areas below Dayton rather than from those above. Of course, this happens more frequently as the distances below Dayton increase. During the storm of July, 1915, the crest at Hamilton was caused primarily by the flood runoff from the Seven Mile and Four Mile Creek valleys. In this instance the Hamilton crest occurred about 10 hours before the maximum stage was reached at Dayton.

The forecasting methods can probably be made plainer by describing a typical case. The flood of January 5, 1917, will be taken for this purpose. On the morning of January 5 the reports showed that the rainfall had averaged about 1.7 inches over the drainage area of the Stillwater River above West Milton, about 1.6 inches over the drainage area of the Miami River above Piqua, and about 1.4 inches over the drainage area of the Mad River above Springfield. For the Stillwater Valley it was estimated that 0.7 of an inch would be required to fill the soil and surface storage; and that the remaining inch, which would run off, would reach West Milton at a maximum rate of about a half an inch in 24 hours, or 7800 second feet. The rating table for the channel at West Milton showed that this would correspond to a stage of 8.0 feet. From the available records of previous floods it was estimated that this stage would be reached at about 3 p.m. By similar methods, making proper allowances for differences in topographical and geological conditions, it was estimated that a maximum rate of runoff of about 10,400 second feet, corresponding to a stage of about 7.0, would occur at Piqua at about 3 p.m.; and that a maximum rate of runoff of about 4,600 second feet, corresponding to a stage of about 7.5, would occur at Springfield at about 2 p.m.

From the known distances of these stations above Dayton and from the known slopes and velocities of the streams it was next estimated that the crest from the Stillwater River would reach Dayton first, that the crest from the Mad River would arrive next, and that the crest from the Miami River would arrive last. Since the quantity of water flowing in the Miami River was about a third greater than that in the Stillwater River and more than twice as great as that in the Mad River it was estimated that the maximum stage at Dayton would be reached when the crest flow from the Miami arrived, and that this would occur

at about 5 a.m. on the following morning. As the rainfall had been comparatively general over the entire valley it was estimated that the runoff from the drainage area below West Milton, Piqua, and Springfield would be more than sufficient to fill the storage space in the channels below the stages corresponding to the expected discharges. However, since the Stillwater and Mad Rivers would both be falling when the crest from the Miami arrived, it was estimated that the maximum discharge at Dayton would be about 21,000 second feet, or slightly less than the combined maximum discharges at West Milton, Piqua, and Springfield. This quantity corresponded to a stage of 8.9 feet and therefore a stage of 9.0 feet was predicted.

From crest relations during previous floods, it was then estimated that a stage of 10.0 feet would be reached at Miamisburg at 6 a.m., January 6; and that a stage of 9.0 feet would be reached at Franklin at 7 a.m. of the same day.

Forecasts were not being made for the dams at that time as construction work had not been started. At Hamilton the crest was caused by Seven Mile and Four Mile Creeks and was nearly reached at the time the forecasts were being made.

The following table compares the forecasted and actual conditions and also shows the time interval in hours between the time the forecasts were made and the time the crests were reached.

Table 41.—Forecasted and Actual Conditions During the Flood of January 5, 1917

Station	Forecast at 10 a. m. January 5			Actual			Advance Warning in Hours
	Stage	Time	Date	Stage	Time	Date	
West Milton	8.0	3 p.m.	Jan. 5	6.5	5 p.m.	Jan. 5	7
Piqua	7.0	3 p.m.	Jan. 5	5.9	5 p.m.	Jan. 5	7
Springfield	7.5	2 p.m.	Jan. 5	7.3	1 p.m.	Jan. 5	3
Dayton	9.0	5 a.m.	Jan. 6	9.1	1 a.m.	Jan. 6	15
Miamisburg	10.0	6 a.m.	Jan. 6	9.5	7 a.m.	Jan. 6	21
Franklin	9.0	7 a.m.	Jan. 6	9.2	9 a.m.	Jan. 6	23

The above described example is probably typical of the forecasting work in this valley. The predictions are not always so accurate for Dayton and Franklin; neither are they always so inaccurate for West Milton and Piqua. As a general rule, however, the forecasts for the locations on the Miami below the Stillwater and Mad Rivers are more certain than those for the upper stations.

APPENDIX

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The following bibliography contains references to the more valuable articles consulted in the preparation of this report. The literature on rainfall, runoff, evaporation, and related subjects is so voluminous that only the more important can be noted. As a rule references to articles dealing solely with rainfall, or stream flow, or the methods of measurement of either, have not been included. Some of the publications referred to have already been cited in the text.

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STATE OF OHIO

THE MIAMI CONSERVANCY DISTRICT

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by

F. L. CAVIS
Chief Accountant

TECHNICAL REPORTS
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6315 630

PREFATORY NOTE

This volume is the ninth of a series of Technical Reports issued in connection with the planning and execution of the notable system of flood-protection works built by the Miami Conservancy District.

The Miami Valley, which forms a part of the large interior plain of the central United States and comprises about four thousand square miles of gently rolling topography in southwestern Ohio, is one of the leading industrial centers of the country. Out of the great flood of March, 1913, which destroyed in this valley alone over 360 lives and probably more than $100,000,000 worth of property, there resulted an energetic movement to prevent the recurrence of such a disaster by protecting the entire valley by one comprehensive project. The Miami Conservancy District, established in June, 1915, under the newly enacted Conservancy Act of Ohio, became the agency for securing this protection. On account of the size and character of the undertaking, the plans of the District were developed with more than usual care.

The plan for flood protection included the building of five earth dams across the valleys of the Miami River and its tributaries to form retarding basins, and the improvement of several miles of river channel within the towns and cities of the valley. The dams contain 7,850,000 cubic yards of earth; their outlet structures contain over 160,000 cubic yards of concrete; the river channel improvements involved the excavation of 4,500,000 cubic yards, and the whole project cost about $30,000,000.

Accounting and cost keeping procedure plays a vital part in the development of any industrial or construction project. Familiarity with the fundamentals of such procedure is essential to the engineer as well as to the business man. Details of practice must, of course, vary with the requirements of each undertaking, but the underlying principles vary but little, and it is believed that a record of the system developed by the District is of such value that it should be made available to the public.

This report describes the structure of such a system. Tucked away amid the conventional statements and written in between the lines, is another story. It is not the purpose of this report to tell that other story, interesting as it may be, but its existence must be understood before the full meaning of what is written here can be appreciated.

The readers of this report must not assume that the mechanism described here worked smoothly when it was first put together. Considerable oiling and adjusting was necessary. The reason was simple. The construction organization was assembled overnight, as most such organizations are. It was composed of

especially high-grade men from many different jobs. Many of the men in subordinate positions had been accustomed to having responsible charge of work. In common with many engineers and contractors, most of them tolerated accounting as a necessary evil. Few had an adequate conception of the principles of the science, but all had strong opinions as to how the job should be handled, and these ideas ranged from the two hooks (one for paid bills, the other for unpaid bills) to the most complicated system of red tape. Before the accounting outline, also new, could be explained to the organization, a great mass of supplies and equipment was delivered, and the work started with a rush.

The oiling consisted mainly in selling the system to the organization, in overcoming dislike of "red tape," and in similar educational work. The adjustments were minor ones, as the main outline was well followed throughout. But in some unexpected places the original mechanism failed to work, and new methods had to be devised. Sometimes these substitutes were clumsy, and served only until better ones could be worked out.

Most industrial organizations are the result of growth, more or less slow, and men and methods grow with them. But a construction organization, put together to do a big job, does not take hold of a ready-made accounting system and start operating it without some friction. The District hopes to aid those that come after, with this presentation of an accounting system which has had unworkable features taken out of it under the pressure of actual operations.

CHAS. H. PAUL,
Chief Engineer.

Dayton, Ohio, November, 1922.

THE TECHNICAL REPORTS

The Technical Reports are published in volumes, called Parts, size 6 x 9 inches, bound in paper covers, and are sold at nominal prices. They are being issued by the Miami Conservancy District in response to the demand for information concerning the flood control works now nearing completion and concerning the results of the investigations conducted by its engineering staff.

More than usual care has attended the preparation of the plans for the thirty-million-dollar flood control works for the Miami Valley. Some of the problems required study and research work of an exhaustive character. The data published is in some respects entirely new, and in other respects is of a kind not available in convenient form elsewhere.

The reports already issued, and which are still available for distribution, are as follows:

PART I—THE MIAMI VALLEY AND THE 1913 FLOOD

By Arthur E. Morgan,
Chief Engineer of The Miami Conservancy District.

128 pages, 6 x 9 inches, 44 illustrations, paper covers; price, 75 cents net, postpaid.

This volume is introductory to the series of Technical Reports, but is not in itself, however, of a technical nature. It contains a description of the flood of March, 1913, which caused great damage in the Miami River Valley, and as a result of which the flood control works now being built were undertaken.

PART II—HISTORY OF THE MIAMI FLOOD CONTROL PROJECT

By C. A. Bock,
Division Engineer of The Miami Conservancy District.

194 pages, 6 x 9 inches, 41 illustrations, paper covers; price 50 cents net, postpaid.

This is a recital of events of interest in the history of the flood prevention movement in the Miami Valley. Its chief value lies in the fact that it describes the steps necessary and the difficulties usually encountered in initiating and carrying out a great public engineering enterprise of this nature.

PART III—HYDRAULIC JUMP AND BACKWATER CURVES

By Sherman M. Woodward, R. M. Riegel and J. C. Beebe.

111 pages, 6 x 9 inches, 88 illustrations, 12 tables, paper covers; price 50 cents net, postpaid.

The first part of the book, entitled "Theory of the Hydraulic Jump and Backwater Curves," sets forth the theory of these complex phenomena, and is written by S. M. Woodward, Professor of Hydraulics and Mechanics at Iowa University, and Consulting Engineer for The Miami Conservancy District.

PART IV—CALCULATION OF FLOW IN OPEN CHANNELS

By Ivan E. Houk,
Hydrographer of The Miami Conservancy District.

283 pages, 6 x 9 inches, 79 illustrations, 48 tables, paper covers; price 75 cents net, postpaid.

This report describes how the maximum flow of the Miami River and its tributaries during the disastrous flood of March, 1913, was deter-

mined by subsequent measurements of the river channels and flood elevations. A large part of the volume is devoted to a discussion of different formulas, many of them of foreign origin, used in computing stream flow.

PART V—STORM RAINFALL OF THE EASTERN UNITED STATES

By the Engineering Staff of The Miami Conservancy District.
310 pages, 6 x 9 inches, 114 illustrations, 11 tables and appendix, paper covers; price 75 cents net, postpaid.

This book sets forth the results and investigations made to reach safe conclusions as to the probable size and frequency of severe floods in the Miami River. Its most noteworthy feature is a study of the relations between the areas covered by different depths of rainfall and the duration of such rainfall for each of the 33 largest storms on record in the eastern half of the United States.

PART VI—CONTRACT FORMS AND SPECIFICATIONS

By the Engineering Staff of The Miami Conservancy District.
192 pages, 6 x 9 inches, 3 diagrams, paper covers; price 50 cents net, postpaid.

This volume has been issued to meet the continued demand for copies of the standard specifications and contract forms in use by The Miami Conservancy District, in the construction of the large earth dams and the protection works along the river channels.

ATLAS OF SELECTED CONTRACT AND INFORMATION DRAWINGS TO ACCOMPANY PART VI.

10 pages of text, 15 x 11 inches, 139 plates, paper covers; price $1.50 net, postpaid.

The Atlas contains 139 representative selections of plans made preparatory to the letting of contracts.

PART VII—HYDRAULICS OF THE MIAMI FLOOD CONTROL PROJECT

By Sherman M. Woodward,
Consulting Engineer for The Miami Conservancy District.
344 pages, 6 x 9 inches, 126 illustrations, 37 tables, index, paper covers; price, $1.00 net, postpaid.

This volume makes an orderly presentation of the methods followed in determining the type of the flood control plans and of the methods of working out the application of the plan. If such a presentation of the subject had been available when this project was undertaken, its use would have saved a very large amount of time and effort.

PART VIII—RAINFALL AND RUNOFF IN THE MIAMI VALLEY

By Ivan E. Houk,
Hydrographer of The Miami Conservancy District.
About 220 pages, 6 x 9 inches, 51 illustrations, 40 tables, paper covers; price, 75 cent net, postpaid.

This report is a record of the results of the studies of the Miami Conservancy District on rainfall and runoff. A considerable part of the volume deals with experiments with both natural and artificial rainfalls.

PART IX—ACCOUNTING AND COST KEEPING OF THE DEPARTMENT OF ENGINEERING AND CONSTRUCTION

By F. L. Cavis
Chief Accountant of the Miami Conservancy District.
112 pages, 6 x 9 inches, 31 illustrations, paper covers; price 75 cents net, postpaid.

Checks should be made in favor of The Miami Conservancy District, and all orders and communications should be addressed to:

THE MIAMI CONSERVANCY DISTRICT,
Dayton, Ohio.

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CHAPTER I

DESCRIPTION AND PURPOSE OF THE WORK

A complete description of the work of the Miami Conservancy District and of the plan of the organization is contained in the Official Plan and the Technical Reports published by the District. Only such description will be given here as is necessary to give a clear understanding of the accounting features.

The work, extending over a period of nearly five years, required the construction of flood-control works on the Miami River and its tributaries from about six miles above Piqua, to Hamilton, Ohio, a distance of about eighty miles. Retarding basins have been provided by the construction of five dams located as follows: (See map on page 16.)

1. Englewood Dam on the Stillwater River about ten miles northwest of Dayton.
2. Huffman Dam on Mad River about five miles east of Dayton.
3. Taylorsville Dam on the Miami River about ten miles north of Dayton.
4. Lockington Dam on Loramie Creek about six miles north of Piqua, Ohio.
5. Germantown Dam on Twin Creek just west of Germantown, Ohio.

All of these basins, except Germantown, are above Dayton. Their purpose is to intercept the water at flood stage, allowing only such volume to pass as can be carried without danger in the channels below.

The work also included extensive improvements in the river channels by widening and deepening, and by the construction of levees, concrete retaining-walls, and revetment, in the cities of Piqua, Troy, Tippecanoe, Dayton, West Carrollton, Miamisburg, Franklin, Middletown, and Hamilton.

The plan adopted made necessary extensive relocations of the railroads passing through the valley. The Cleveland, Cincinnati, Chicago & St. Louis Railroad, the Erie Railroad, the Baltimore & Ohio Railroad, and the Ohio Electric Railroad lines traversed the retarding basins. Numerous other public utilities were affected. Telegraph, telephone, and power lines, sewer and water systems, and in one instance an entire town, had to be removed from the basins and from locations along the rivers where they interfered with the construction of the works or encroached upon the basins and enlarged channels. The dams are all of earth and gravel, constructed by the hydraulic fill method. Each one has outlet works of concrete and a concrete spillway. Most of the river channel excavation and levee construction was done by large excavating machines of the dragline type. A description of the construction plant and detailed designs of the various structures will be found in the various District publications.

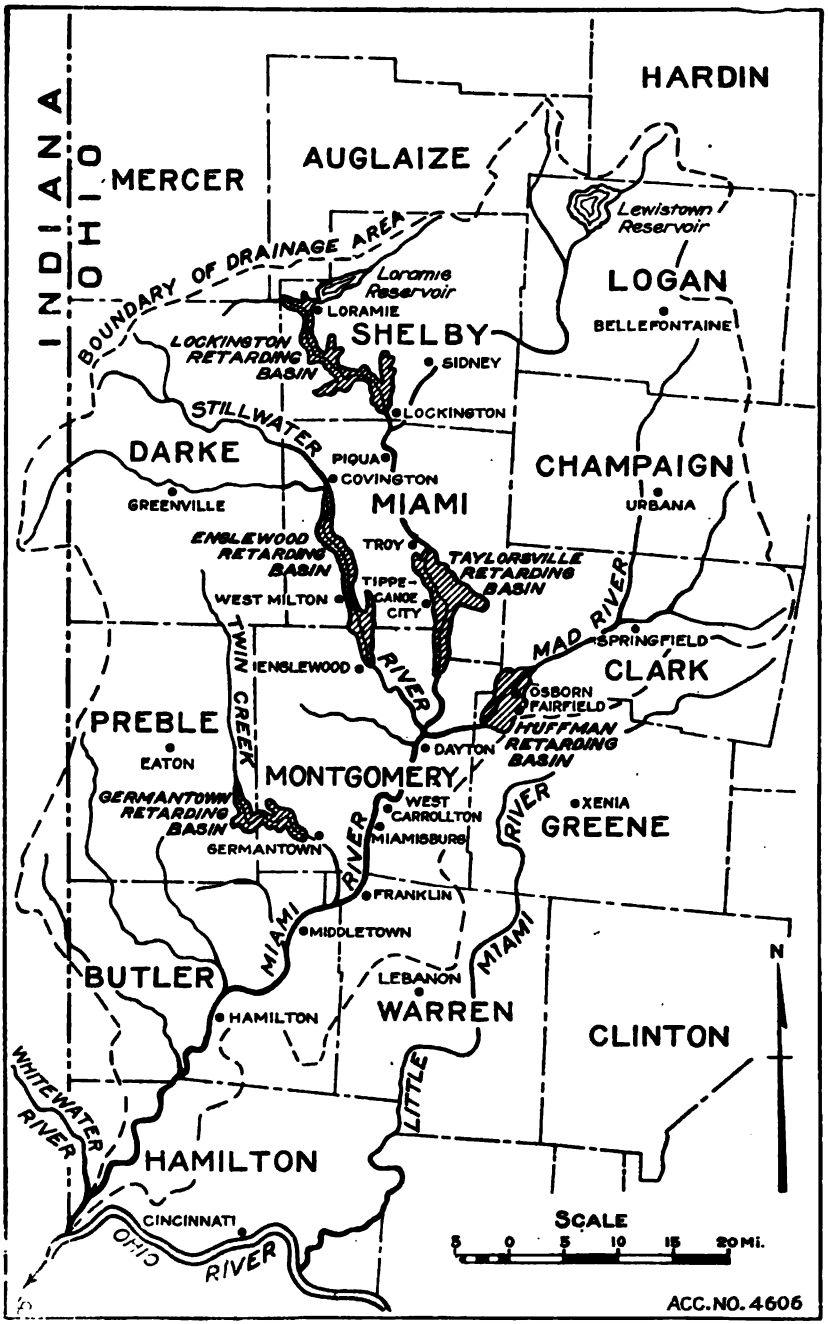


FIGURE 1—MAP OF THE MIAMI VALLEY

The shaded portions represent the basin areas covered by water if a storm large enough to fill the basins should occur. The watershed area above Hamilton is 3672 square miles.

It was the original intention to do all the construction work under contract, and the principal divisions of work were advertised for bids in the fall of 1917. The entry of the United States into the World War and the disturbance of labor and industrial conditions made such procedure impracticable, however. Under the conditions existing, it was not to be expected that any dependable bids would be made for work of such magnitude. Bids were opened in November, 1917, and all except one were considered to be unreasonably high, or were irregular. The one accepted bid was for a comparatively small section of levee construction in Dayton.

The District, thereupon, proceeded to develop a construction force as a part of its own organization, to secure the necessary equipment and go ahead with the work. The engineering force was already well organized. A construction manager was selected to serve as the head of the construction organization, and for each of the larger divisions of work a superintendent was appointed. Machine shops, warehouses, and other necessary features were established. The construction organization was very similar to that which would have been developed by a contractor, and the relation between the engineering and construction forces was much the same as would have existed under contract work. Specifications had, of course, been prepared before advertising for bids, and those specifications were followed in carrying out the work, such modifications being introduced as were demanded by the change in procedure.

The two branches of organization mentioned—engineering and construction—together with the necessary accounting division, purchasing division, paymaster's staff, etc., constituted the Department of Engineering and Construction, and it is with the system of accounting and cost keeping employed for that department that this report has to deal. In this report only such mention will be made of the administrative department as is necessary to explain the method of tieing in the records of the Department of Engineering and Construction with the general accounts of the District as kept by the treasurer.

The Land Depa·tment also handled its own detailed accounts affecting land, rights-of-way, and farming operations, a control account being carried in the secretary-treasurer's office. The accounts for the legal, taxation, and other departments were kept in detail in the secretary-treasurer's office. The secretary-treasurer's office was referred to as the Administrative Department and the accounts for all departments were controlled by general accounts kept by the treasurer, but these carried no detail and consisted for the most part of a single account for each department, disbursements being charged and collections credited thereto.

The Department of Engineering and Construction operated as a unit, close coöperation between the engineering and construction branches being maintained. All the various features

of the work were under direct control of the chief engineer and construction manager, but each of the larger divisions was operated, to a considerable extent, as a separate business unit. Immediate responsibility for the affairs of a feature rested with the division engineer and superintendent appointed for that feature. They reported to the chief engineer and construction manager respectively. Plans for carrying on the work and transfers of equipment, supplies, etc., from the general warehouse to a feature or from one feature to another, were conducted with a general attitude of maintaining each feature as a separate business unit as far as it was possible to do so without sacrificing the greater good of the job as a whole. The accounting system recognized each feature as a unit.

By far the greater part of this work was done by the District's own forces. Except for the public utility work, the contracts that were let were for comparatively small amounts.

It can be readily understood that almost every activity in connection with the art of construction entered into the District's work. The accounting, extensive at best, became more complex by reason of the incidental operations necessary for construction by force account and the accommodation of employees. Such operations were:

1. Building and maintaining camps.
2. Operation of boarding houses.
3. Operation of stores.
4. Operation of warehouses.
5. Operation of garage and motor vehicles.
6. Operation of machine and electrical shops.
7. Gravel washing and screening plants.

An accounting system that will adequately record the financial transactions for all of such operations must be very broad and flexible and at the same time provide facilities for cost finding in sufficient detail to enable intelligent comparison of results on different jobs while the work is in progress. Expense of keeping cost is not justified unless results can be used as a check on current operations. Therefore, one of the most important features of a cost system is to have the figures at hand as the work progresses.

In the work of the District almost all the elements entering into commercial accounting, except profit and loss, were involved. As the question of profits did not enter into the District's work, all expense was absorbed into feature cost, and any revenue derived from incidental operations was credited to construction cost.

This report concerns the accounting and cost keeping as actually conducted and the system is not presented as a perfect model. Facts are set forth as accurately as possible and in some instances discussion is offered with the thought of suggesting improvements in the procedure.

CHAPTER II
ORGANIZATION AND ACCOUNTING PLAN

The accounting work was centralized as much as possible, most of the detail work of assembling cost being done in the headquarters office at Dayton. The force consisted of a chief accountant, who had general charge of all the work of accounting in the department, assisted by bookkeepers and clerks, the force varying according to the volume of work during different stages of construction.

At each of the large units of construction there was a field office and a clerical force consisting of a field clerk, one or more timekeepers, and assistants as needed. Except in the case of the Dayton local protection work, field warehouses were also established under the direct charge of a warehouse man. During rush seasons an assistant was required in some of the field warehouses. The field clerk had general supervision over all accounting work on his feature, and was responsible for the proper assembling of cost data and forwarding to the accounting division in Dayton. He reported to the division engineer in charge of his feature and the timekeepers and warehouse men in the field were immediately responsible to the superintendent of construction, but all field clerical men were subject to orders of the chief accountant in accounting matters.

The general warehouse, machine shop, electrical shop, and garage were located in Dayton. Each of these was under the supervision of a superintendent reporting to the chief engineer and construction manager, but the clerical force, while under the immediate control of the superintendent, was subject to the direction of the chief accountant in accounting matters.

In this respect the clerical force was a combination of a line and staff organization, and received orders from two sources. But this did not prove objectionable; in fact it worked out to considerable advantage, as it relieved the field officials of much detail in connection with the accounting work and allowed them to devote their entire time to construction problems. Nevertheless, care was taken to avoid petty interference with the field and to allow superintendents and division engineers full authority over their men.

The field accounting was standardized and the headquarters supervision was principally to see that it was kept up to the standard and to advise the field forces when special problems came up. There was usually someone in the headquarters office who could be assigned to a feature temporarily to help bring back work up to date, or assist in emergencies.

Direct telephone lines ran from the switchboard at headquarters to the five dams, and to the warehouse, shop and garage in Dayton. Excellent long distance connections were to be had to the other features, and much of the business with the field was transacted over the telephone, reducing correspondence to a minimum. Papers, invoices, blotters and all other mail was sent in to headquarters daily. The field men were encouraged to tell their accounting troubles to headquarters, and careful consideration was given to all questions, however trivial.

CHAPTER III

PHYSICAL DIVISIONS, THE ACCOUNT NUMBER SYSTEM, AND CLASSIFICATION OF COST

In work of any magnitude where a large number of accounts are involved, some system of designating physical units and details of accounts under these units, must be adopted to facilitate assembling of cost data.

Before any construction was undertaken, all the physical divisions were well defined in the Official Plan prepared under authority of the Conservancy Law, and complete specifications were prepared. In the Official Plan the physical divisions (called features) were numbered, and numbers were also assigned to the items of work for each feature. These numbers formed the basis for the account number scheme when the accounting system was outlined. A knowledge of this plan of symbolizing the different features of the work is essential to a thorough understanding of the discussion of accounting details which follows, so the account number scheme is given in full below.

Physical divisions, or features, which form one unit of construction were the first units in the classification of cost. They were the natural subdivisions which would have been made for separate contracts in case the work had been let by contract.

Feature Number	Name
1	Germantown Dam and Road No. 1
2	Englewood Dam and Roads 3, 4, and 5
3	Lockington Dam and Roads 8 and 9
4	Taylorsville Dam and Roads 12 and 13
5	Huffman Dam and Roads 16 and 17
6	Road No. 2, Germantown Basin
7	Road No. 6, Englewood Basin
8	Road No. 7, Englewood Basin
9	Road No. 10, Lockington Basin
10	Road No. 11, Lockington Basin
11	Road No. 14, Taylorsville Basin
12	Road No. 15, Taylorsville Basin
16	Highway Bridge at Germantown
17	Highway Bridge at Englewood
18	Highway Bridge at Lockington
19	Highway Bridge at Mill Creek Spillway
20	Highway Bridge at Taylorsville
21	Highway Bridge at Huffman
24	Raising Bridge over Brush Creek
25	Moving Bridge over Stillwater River
27	Tippecanoe City Local Protection
28	Tippecanoe City Drainage and Sewer Pumping Station
31	Tippecanoe City Substructure, Water and Light Station
35	Piqua Local Protection

Feature Number	Name
37	Troy Local Protection
41	Dayton Local Protection above Island Park
42	Dayton Local Protection below Island Park
49	West Carrollton Local Protection
50	Miamisburg Local Protection—East Side
53	Franklin Local Protection—East Side
55	Middletown Local Protection—General
58	Hamilton Local Protection—Miami River
60	Hamilton Local Protection—Drainage
61	Black Street Bridge at Hamilton
62	Adams Street Bridge at Troy
63	Market Street Bridge at Troy
64	Main Street Bridge at Piqua
100	Preliminary Investigations
200	General Expense—Headquarters Office
1001	Relocation Big Four and Erie R. R., Dayton to Enon
1002	Relocation Ohio Electric R. R.
1003	Relocation Baltimore & Ohio R. R., Dayton to Tippecanoe City.
1004	Big Four R. R. in Lockington Basin
1005	Relocation of Telegraph and Telephone Lines
1006	Miscellaneous Damages to Public Utilities
1007	Tates Point Crossing of Big Four and B. & O. R. R.
1011	Piqua Municipal Improvements and Public Utilities
1012	Troy Municipal Improvements and Public Utilities
1014	Dayton Municipal Improvements and Public Utilities
1015	Miamisburg Municipal Improvements and Public Utilities
1016	Franklin Municipal Improvements and Public Utilities
1017	Middletown Municipal Improvements and Public Utilities
1018	Hamilton Municipal Improvements and Public Utilities

The next subdivision of cost was by classes of work under the feature. The different kinds of work to be done and of materials to be furnished were classified into items, an item meaning a certain kind of work to be done, or material to be furnished, in a certain definite manner, regardless of its location. Each item was designated by a name and number, the numbers running from 1 to 90. In Part VI of the Technical Reports will be found complete description of all items and of the items pertinent to each feature or contract. Not all items, of course, were found under each feature, but, wherever found, the number assigned for any item meant the same class of work or material, with the result that uniformity in classifying expense was obtained and common reference to items was by numbers rather than names. The following table gives the item numbers and names that were used in assembling cost:

Item No.	Name
1	Stream control at dams
2	Clearing and grubbing
3	Removal of buildings
4	Trimming and shaping slopes
5	Soil stripping
6	Excavation, new channels, cut-off trenches, etc.
9	Excavation, river channels
13	Excavation, outlet works and spillways
16	Excavation, minor drainage
19	Excavation, highway cuts and fills
22	Excavation, retaining walls, bridge piers, etc.
25	Excavation, backfilling trenches
31	Embankment, hydraulic fill
32	Embankment, semi-hydraulic fill
33	Embankment, rolled fill
34	Embankment, levees, material from channel excavation
35	Embankment, levees, material from borrow pits
36	Embankment, miscellaneous
37	Backfilling
38	Surface dressing and grassing
39	Waterproofing concrete in bridges
40	Concrete, bridge railings
41	Concrete, walls, piers, outlet works, spillways, etc.
42	Concrete, in bridge arches
43	Concrete, in spandrel walls and counterforts
44	Concrete, monolithic revetment
45	Concrete, flexible slab revetment
46	Concrete, in culverts, bulkheads, gate chambers
47	Concrete, in sewers, manholes, etc.
48	Stone masonry
53	Drilling foundations
54	Grouting foundations
55	Furnish and lay vitrified pipe
57	Dry rubble paving
58	Riprap
59	Gutter paving
60	Crushed rock or gravel
61	Rolled gravel on highways
62	Water-bound macadam
63	Bituminous-bound macadam
64	Brick paving on concrete foundations
65	Concrete paving
66	Miscellaneous street paving
67	Guard rails
68	Wire fences
69	Lighting system for bridges
71	Timber and lumber for grillage
72	Timber piles
73	Wood sheet piling
74	Steel sheet piling

Item No.	Name
75	Furnish and lay cast iron pipe
77	Furnish and place flood gates
78	Steel highway bridges
80	Lowering track at Englewood bridge (D.C. & P.R.R.)
81	Raising, moving, and re-erecting bridges
82	Miscellaneous cast iron, wrought iron, and steel
84	Wire strand for flexible slab revetment
85	Cleaning up
86	Pumping stations
87	Pumping equipment
88	Removing and re-erection of existing pumping and power stations
90	Extra work

The specifications provided for more items than shown in the above classification. For example, each kind of excavation was divided into Class 1, Class 2, Class 3, and sometimes overhaul. Concrete was also subdivided into great detail and a separate account provided for reinforcing steel. As work progressed, it was found impracticable to keep accurate cost of these different classes, so for accounting purposes combinations were made.

An account number was not complete unless it contained both a feature and item number. Thus number 2-41 meant concrete in piers, walls, outlet works at the spillway on feature 2, which was the Englewood Dam. The same kind of concrete on any other feature was represented by the same detail number (41), which, when preceded by the appropriate feature number, indicated on what feature the work was done.

For Preliminary Investigations and General Expense (Accts. 100 and 200), special classification was made as follows:

Preliminary Investigations

101	Surveys and Reconnaissance
102	Estimates and preliminary designs
103	Expert engineering
104	Experimental work
105	Test pits, borings, etc.
106	Hydrography
107	Office expense
108	Legal work on Official Plan

General Expense

201	Consulting services
202	Engineering, Headquarters Office
203	Engineering, Field Offices
204	Inspection
205	Accounting
206	Miscellaneous clerical
207	Testing Laboratory
208	Miscellaneous office supplies and expense
209	Hydrography and flood warning
210	Photography
211	Taxes and miscellaneous
213	Employment

These accounts for general expense were used for features as well as for expense of headquarters office. The account number of the feature followed by the item number was used for job overhead; thus field engineering at Lockington would be charged to 3-203, while engineering of a general nature not properly chargeable to any particular job would be charged to 200-202.

A special classification was also used for railroad relocation and public utilities, as classes of work entirely different from dam and river work were encountered. Letters were used in this case to avoid confusion of the accounts with other feature items. The classification follows:

A	Engineering and inspection
B	Grading (by contract)
C	Track laying and ballasting
D	Right-of-way
E	Excavation for structures
F	Concrete in structures
H	Ballast (M. C. D.)
J	Temporary railroad yards
K	Miscellaneous
M	Rail and accessories
MM	Unloading rail
MMM	Rehandling rail
N	Ties
NN	Unloading ties
NNN	Rehandling ties
P	Right-of-way fence
Q	Crossing gates
R	Signals
S	Excavation and fill for road bed (M. C. D.)
T	Railroad stations
W	Revenue from betterments (Credit Account)
Z	Salvage of old material (Credit Account)

Owing to the nature of the relocation work, detailed specifications were not worked out in advance to the same extent as for the other work, so the classification of cost was developed as the work progressed and subdivisions of some of the items were made by assigning other letters or combinations of letters. For example, when it was desired to keep a record of the cost of unloading and rehandling rail, instead of putting all expense in M, the unloading was assigned account MM and rehandling MMM. The railroad symbols proved somewhat cumbersome and results obtained were not as satisfactory as for the other features.

The detail into which cost should be classified is a much-discussed question about which there is a great divergence of opinion. The outline shown above was adopted for the Miami Conservancy District as the official classification. Some of the detail kept was of no real practical value to the construction officials as the work progressed, although it has a certain historical value. On the other hand, there was a demand for more analytical cost for some items of work, such as hydraulic fill and concrete. Intelligent comparisons, with explanations of reasons for dif-

ferences, cannot be made from the information shown, and the lack of this detail led to the compilation of supplemental field cost by field offices. For example, hydraulic fill was kept by field clerks and timekeepers, under the direction of division engineers, in about the following classification:

Hydraulic fill, Feature 2 (Englewood), Item 31

(a) Labor
  1 Excavation
  2 Transportation
  3 Track work
  4 Pumping
  5 Levees and slopes
  6 Miscellaneous
(b) Power
  1 Excavation
  2 Transportation
  3 Pumping
  4 Levees
  5 Supplies
(c) Supplies
(d) Plant
(e) Dumps and hog box installation
(f) Camp
(g) Dredge pipe
(h) Pumps and runners
(i) Industrial insurance
(j) Shop work (Repairs)
  Subtotal
(k) Job overhead (Engineering, inspection, etc.)

This detail was not continuously maintained. It was intended for temporary use of division engineers as a test from time to time and covered for the most part direct expense only.

The tendency is always to provide too much detail, with the result that the finer subdivisions of cost are inaccurate and the books are encumbered with such a mass of accounts that it is difficult to get out reports in time to be of value on current work. It was for this reason that the general books carried the classification of cost to item numbers or kinds of work only. The principal objection to informal cost records is that, owing to lack of coördination with the general and financial books, some items may be overlooked and a unit cost arrived at that may be misleading. In the present instance, however, it was possible to compare the total unit cost of a given item as reported by the field records with the accounting division's report of cost of the same unit, and as a rule these compared very closely.

So far, classification of cost for features or jobs only has been discussed. Ultimately, of course, all cost was absorbed in features, but all cost could not be applied direct to its ultimate destination. Like the manufacturing expense of a factory, a large percentage of the cost had to be first assembled as a part of some incidental operation or production unit, like a gravel plant, or camp maintenance, depreciation on plant, or other items of over-

head. However, a much greater portion of cost on construction of this character can be charged direct than in factory cost finding.

Therefore, methods had to be devised for assembling the cost of plant and equipment and the operation of shops, stores, messes, etc. An explanation of such accounts and of the classification used in keeping the costs for them, follows.

Plant Accounts. By plant is meant equipment and machinery installed into a working unit. Great expense was incurred in the erection of camps for the accommodation of employees and for equipment and its installation, before a yard of material was excavated from the river or placed in the dams. If this were a permanent installation, such expense would be a charge against capital and renewed from time to time. Such was not the case in the work of the District, and the cost of all plants had to be absorbed into feature cost by the time of completion of the work. This was accomplished by subdividing plant and equipment into classes according to function, assembling first cost into each of these accounts, and charging it out to features gradually as work progressed. The principle was the same as for charging depreciation on factory equipment. Depreciation methods are discussed elsewhere in this report (page 67), so the present chapter will deal with classification of cost only.

The following tables show the plant accounts provided and the account numbers or symbols used for the subdivision of accounts. In order to avoid conflict with feature item numbers, plant and operating account numbers began with 0. Numbers 01 to 050 were reserved for plants. Separate plants were provided for each job and the complete account number consisted of the feature number followed by the plant number. Further division of plant cost was accomplished by adding letters for items under each plant.

01 Camp Construction
- A Mess, store and community buildings
- B Shops, warehouses, garages, etc.
- C Dwellings, bunkhouses, and quarters for employees
- D Public service (water, gas, lighting, heating, telephone, sewers, fire prevention, etc.)
- E Camp roads, pavements, open drainage, temporary bridges, etc.
- P Dismantling
- Y Depreciation (Credit account)
- Z Salvage (Credit account)

02 Concrete Plant
- A Mixers (includes motors, boilers, etc.)
- B Permanent forms
- C Screening and washing equipment
- D All other equipment
- G Extraordinary repairs
- M Hauling and erection
- P Dismantling
- Y Depreciation (Credit account)

03 Construction Plant, General (For large equipment not otherwise classified)
- A Pumps (for unwatering)
- B Pile drivers
- C Derricks and hoists
- D Cableways
- E Job lighting
- F Other equipment
- G Extraordinary repairs
- M Installation
- P Dismantling
- Y Depreciation (Credit account)

04 Excavation Plant
- A Excavation machinery (all types)
- B Hauling equipment (locomotive, cars, etc.)
- C Trackage
- D Tools, buckets, etc.
- G Extraordinary repairs
- M Hauling and installation
- P Dismantling
- Y Depreciation (Credit account)

05 Hydraulic Pumping Equipment
- A Dredge pumps and accessories
- B Water pumps and accessories
- C Monitors and giants
- D Pipe, trestle, flumes, etc.
- G Extraordinary repairs
- M Installation
- P Dismantling
- Y Depreciation (Credit account)

06 Miscellaneous Equipment and Tools
- A Digging tools (small scrapers, picks, shovels, etc.)
- B Furniture and office equipment
- C Instruments and engineering equipment
- D Mess equipment and utensils
- E Boots and other personal equipment
- F Store fixtures
- G Electrical shop equipment and tools
- H Machine shop equipment and tools
- J Farming equipment and tools
- M Camp equipment
- N Garage equipment and tools
- P Dismantling
- R Shop patterns
- S Warehouse equipment and tools
- Y Depreciation (Credit account)

07 Motor Vehicles
- A Passenger cars
- B Trucks
- C Motorcycles
- D Extra parts in use
- G Extraordinary repairs

P Dismantling
Y Depreciation (Credit account)

08 Temporary Railroads
A Roadbed (including structures)
B Trackage
C Equipment not in other plants
G Extraordinary repairs
P Dismantling
Y Depreciation (Credit account)

011 Transmission Power Lines
A Right-of-way
B Labor erecting
C Materials (poles, wire, insulators, transformers, etc.)
D Travel and auto expense
G Extraordinary repairs
P Dismantling
Y Depreciation (Credit account)

Still another class of accounts, "Operating or Clearing" accounts, were different from plants. They took expense which could not be charged direct to features but which was a part of current cost, whereas plant cost was an investment for the whole period of construction, or for the life of the equipment. These operations were again subdivided into two classes: (1) those that were supposed to be self-sustaining, such as messes, stores, etc., and (2) those that were purely suspense accounts, assembling cost for convenience or statistical purposes, and being later "cleared" in some more or less arbitrary manner to feature cost. The handling of both these divisions was similar and they have been grouped together here.

The following is the classification of cost for all operating accounts used. Numbers 051 to 091 were assigned for this group.

051 Camp Operation and Maintenance
A Labor
B Supplies
C Minor repairs to buildings
K Miscellaneous
X Depreciation (from camp construction)
W Revenue from service to employees (Credit account)
Y Distribution (Credit account)

The "X" account took the amount written off monthly from camp construction (01) and in this manner a portion of cost of constructing camps was absorbed into the work with the cost of maintenance. The account was closed monthly by charging the total to features on the basis of direct labor cost.

It may be well to explain that these operating accounts sometimes carried balances over from month to month, either debits or credits. Closing figures were furnished by field clerks when they submitted labor distribution, and the current month's expense for a given operation was estimated from past experience and this amount cleared. The method was approximately correct and saved much time in the general office in making closing entries, thus enabling monthly cost reports to be rendered more

promptly. Rates or amounts of distribution were changed from time to time so as to keep balances as small as possible. At the close of the job, final balances were distributed to feature cost.

052 Mess Gardens
- A Labor
- B Supplies
- C Minor repairs to equipment
- K Miscellaneous
- W Revenue (Credit account)
- Y Distribution (Credit account)

Profit or loss on garden accounts was closed to messes. As a matter of fact, operation of gardens was a failure financially and the plan was discontinued after the first two years.

053 Operation of Messes
- A Labor
- B Foodstuffs
- C Supplies
- K Miscellaneous
- X Depreciation
- W Revenue (Credit account)
- Y Distribution of Loss (Credit account)

Revenue from sale of meals was credited direct to mess operations (Account 053-W) and loss charged monthly to feature cost on a basis of direct labor. This subject is discussed in more detail in a subsequent chapter (page 75).

054 Operation of Motor Vehicles
- A Labor
- B Supplies
- C Gasoline
- D Repairs
- E Tire replacements
- F Garage rental and taxes
- H Lubricants
- K Miscellaneous
- X Depreciation
- Y Distribution (Credit account)

Cost of operation of motor vehicles was transferred to features on a basis of actual service rendered at fixed rates. This is discussed fully in a subsequent chapter (page 73).

055 Operation of Machine Shops
- A Labor
- B Materials for jobs
- C Supplies for shop
- D Rental and taxes
- K Miscellaneous
- X Depreciation
- Y Distribution (Credits on shop orders)

Shops were operated on a job order basis discussed in full later in this report (page 71).

056 Operation of Electric Shop
- A Labor
- B Material for jobs

C Supplies
D Rental and taxes
K Miscellaneous
X Depreciation
Y Distribution (Credits on shop orders)

057 Operation of Stores
A Labor
B Merchandise for sale
C Supplies
K Miscellaneous
X Depreciation
W Revenue from sales

Store accounts were closed upon completion of work only. (See page 75.)

058 Superintendence and Field Accounting
A Construction Manager
B Feature Superintendents
C Field clerical, timekeeping, etc.
D Travel and auto expense
E Office supplies and expense
F Proportion of paymaster's expenses
K Miscellaneous
X Depreciation
Y Distribution (Credit account)

These accounts were closed monthly to feature cost on a basis of direct labor.

059 Operation of Warehouse
A Labor
B Purchasing
C Supplies
D Rental and taxes
E Hauling and handling
F Handling cement (into warehouse only)
G Loss on damaged goods
H Warehouse repairs
K Miscellaneous
X Depreciation on tools
Y Distribution (Credit account)

This account was closed monthly by transfer of total cost to an account in the general ledger designated "Freight and Handling." Ten percent was added to the value of issued materials upon issue to work and was credited to "Freight and Handling" to absorb this cost. (See page 56 for full discussion of warehouse accounting.)

060 Operation of Meat Shop
A Labor
B Foodstuffs
C Supplies
K Miscellaneous
X Depreciation
W Revenue

The shop was operated for the sale of meat to employees.

064 Operation of Gravel Washing Plants.
- A Labor
- B Supplies
- C Repairs
- K Miscellaneous
- X Depreciation
- W Revenue from sale of gravel (Credit account)
- Y Distribution, gravel used on Miami Conservancy work (Credit account)

Fixed rates for the sale of gravel were established by contract and the revenue was applied to the credit of the gravel account. Gravel used on the work was charged to feature items at approximately the cost per cubic yard for production, and the balance, debit or credit, was closed to feature cost when the plant ceased operating.

065 Reconciliation or Adjustment Account
- A Bills of lading open
- C Transfers open
- E Store and mess inventories

For convenience in keeping records, a number in this group was used as a suspense account to which to charge items which could not be immediately allocated to features. For example, in the movement of supplies from one feature to another, the transferring feature took credit when a bill was issued, but until the supplies were accepted by the receiving feature the debit remained in one of these rconciliation accounts (A or C). Merchandise which had been charged to stores and messes and was on hand at the end of a month, was credited to the proper store or mess operating account and debited to 065C. This was immediately reversed upon opening the books for the next month. By this method, actual current cost was shown on monthly reports, regardless of what had been issued to stores and messes and not sold or consumed.

066 Operation of Salvage Division
- A Labor
- B Materials and equipment from features
- C Supplies and repairs
- D Labor and material from shop
- K Miscellaneous and auto service
- M Cost of selling
- N Commissions on sales
- X Credit—percent of sales of feature material
- Y Credit—sale or transfer of equipment

The salvage account was opened when equipment commenced to wear out and old material accumulated. The expense of putting old apparatus into salable or usable condition was charged to this account, and when reclaimed articles were sold or were issued to some feature, the price obtained was credited to 066-Y. The balance in the account was estimated to approximate the value of equipment available for sale, and was closed monthly into general ledger account No. 36 (equipment not in use). All profit or loss was closed to feature cost at completion of the job.

Equipment was taken into this account, as parts of the work were completed, with the result that plant accounts carried only equipment in use. The question of salvage will be discussed more in detail in the chapter on Equipment. (Page 69.)

069 Payroll Advances on Variable Fee Contracts
- A Price Brothers contracts
  - Subdivided to provide an account for each contract
- B Donald Jeffries contracts
  - etc.

A considerable number of small contracts were carried out under a form of agreement which provided for payment by the District of actual cost plus a variable fee which increased or diminished with the decrease or increase of the actual construction cost as compared with an estimated base price cost. A copy of the form of agreement is shown in the Appendix.

Some very satisfactory results were obtained under this variable fee form of contract. In writing such an agreement, great care should be exercised in establishing the method of varying the fee and in the determination of base price figures.

The 069 account was provided to take care of temporary advances on contractors' payrolls, etc., pending preparation of estimates at the close of a month. When these estimates came through, the total earned was charged to feature cost and the amount of advances credited to this expense account.

071 to 091 Dragline Operating Accounts
- A Labor
- B Supplies, expendable (except cable)
- C Repairs (all not charged to Plant)
- D Fuel or power
- E Cable
- K Miscellaneous
- X Depreciation
- Y Distribution (Credit account)

These accounts were used to assemble the cost of operating dragline excavators, a separate account being assigned for each machine by prefixing a number to designate the machine. At the close of a month the total for any excavator was cleared to units of work on which it had been engaged. (Page 78.)

Maintenance Accounts. A feature or unit of the work having been completed and the final cost reported, any subsequent work on that feature was regarded as maintenance. A separate fund was available for such work and the cost was assembled under the following classification. Symbols used were the feature number followed by a letter to indicate the kind of work.

- A Patrolling, watchmen, caretakers, cutting weeds, etc.
- B Clearing trees, brush, and drift above dams and from channels.
- C Seeding or sodding levees and dams
- D Planting trees and shrubs on and near levees and dams
- J Barriers (check dam)
- K Excavation in channels
- L Repairing roads

M Minor repairs to levees and revetment walls
N Minor repairs to dams and other structures
O Operation and maintenance of flood gates
V Engineering
W Proportion of general office expense
Y Credit for sale of gravel from channels

Centralized vs. Field Cost Keeping. In this centralized plan of assembling cost, the original data upon which cost records were based was forwarded in more or less crude form from field offices to the general office at Dayton and there worked into final form. There are many advantages to this plan.

It concentrates clerical work so that the employment of clerks trained in accounting is justified. It relieves field offices of much detail which in some instances takes up too much of the time of engineers and superintendents. It enables the chief accountant to supervise the details of accounting work more closely. It saves traveling expenses of inspectors.

On the other hand, very satisfactory results could have been obtained from cost records if controls only had been kept in the main office and details kept on the jobs. The field costs referred to on pages 24 and 25 as being kept by the division engineers could have been made a part of the official record. The principal reason for keeping cost is to correct expensive practices and effect economies in operations as they proceed. Little good it does to the work in hand to know after it is done that it might have been done cheaper, although the information may have some value as a matter of experience. It was possible to get reports out in about eight to ten days after the close of the month with the centralized plan, whereas if details had been assembled on the job, the reports might have appeared several days earlier, and data would have been at hand to compile cost day by day when it would have been of advantage to do so. So much of the cost of work of this nature is overhead, that complete reports are not practicable more often than once a month, but direct labor and material could have been assembled in such a way that totals could have been known from day to day if necessary. It is possible that in the field cost keeping more work in the aggregate would have been done, but certain field employees were essential (field clerks, timekeepers, warehouse men, etc.), and it is doubtful if the additional force necessary for the cost keeping would have been much more than might have been saved by reducing the central office force. To keep up detail work from day to day, more work will be required in a month, but the time saved at the end of the period by preventing congestion more than compensates for it.

Construction forces look upon cost from the central office as foreign figures over which they have no control, and are apt to question their accuracy. But if the data are compiled by their own employees, in their own offices, no such objection can be raised and they must accept responsibility for them. The distribution to items of cost is likely to be more accurate. The officials

most concerned with the cost (the division engineer and superintendent in immediate charge of the work) have the first access to the results and can apply any corrective measure which the figures reveal to be advisable. It is often hard to get construction men interested in costs. To create interest, responsibility must be created, and by making a superintendent responsible for the compiling of costs he cannot help becoming interested in the results.

The physical conditions and the organization in connection with the work of the Miami Conservancy District furnished almost ideal conditions for keeping field costs. Whether such a plan would have proved more satisfactory than the scheme used, is, of course, problematical, but it would be worth serious consideration in organizing for similar work.

CHAPTER IV

CONTROLLING AND GENERAL LEDGER ACCOUNTS

In the preceding chapter classification of cost accounts was discussed. In this chapter a list of the general ledger and controlling accounts will be given with a brief description of their purpose and relation to the details previously mentioned. The term "controlling account" is used to describe the summation of any subdivision of accounts as kept in the general ledger, detail for which is not carried in the ledger because it would be too cumbersome to handle. The various items entering into the transaction are carried to the account by totals only, the details being shown in a subsidiary or auxiliary book. The total of all balances in the detail book must equal the balance in the general account and in this way the general account serves as a check or "control" over the detail.

General accounts were numbered beginning with 1, and whenever reference was made to any such account the number was preceded by the letters "G. L." to distinguish the account from any feature item or number.

Asset Accounts. Group I—Cash. This was a controlling account for cash collected by field clerks. It was subdivided to show the cash collected by each field clerk but reported in one item on the Balance Sheet. When daily or weekly reports of cash were received, this account was debited and the proper stores or messes credited. When an amount collected was remitted to the treasurer, "Collections" (G. L. 71) was debited and field clerk's cash account credited. The balance was the cash in the hands of the field clerk. Numbers 1 to 9 were reserved for cash accounts.

Group II—Inventories. For inventory accounts and inventory adjustments, numbers 11 to 20 were assigned. A brief description of the procedure under these accounts follows.

No. 15 Reconciliation of Inventories. Sometimes goods would be received ahead of invoice or the price would not be known accurately. The materials might be needed on the job at once, and in order to keep proper control, prices would be estimated. This related more to repair parts than to any other class of material. The regular inventory account for that class of material would be charged with the estimated price at once. When the actual price was obtained, if the difference was small, G. L. 15 would be debited or credited, thereby avoiding the necessity of adjusting prices on stock records or issuing debit and credit slips if the material had been issued. In practice the method worked out very well, as the balance in the account was never but a very small fraction of current business.

No. 16 Warehouse Stock. This was the control for the general and miscellaneous stock in the warehouse. Details were shown on stock cards referred to in the chapter on "Accounting for Materials." The balance was the book value of stock on hand

at any date and was expected to equal the sum of stock card balances. (See page 56.)

A separate account was kept for each feature warehouse.

For the first two years of the District's activities, separate controlling accounts were kept for different classes of material as follows:

No. 11 Cement
No. 11A Cement sacks
No. 12A Vitrified pipe
No. 12B Cast iron pipe
No. 12C Flood gates
No. 13 Coal
No. 14 Lumber
No. 16 General warehouse stock
No. 17 Equipment

This separation was provided to keep a record of the movement of different classes of material, but conditions developed that made this information unnecessary, and as the keeping of so many controls caused much extra clerical work, all inventory accounts were combined with No. 16 early in 1920.

No. 17. After the inventory account for equipment was discontinued, this number was assigned to adjustment account "Inventory Depreciation." Overages in stock were credited and shortages debited to this account. An overage was caused, in the first place, by an under-valuation in inventory in 1919, and it was credited to this account rather than to distribute it back to feature cost. Subsequent drop in the prices of materials justified this action and losses and adjustments by sales of surplus stock at less than cost were debited to the account. In this way it acted as a reserve for depreciation in value of warehouse material. The balance was a credit balance, and indicated the amount held in reserve to meet such losses.

No. 18 Freight and Handling. Carload freight and the transportation charges on bulky articles, such as cement, coal, large equipment, etc., were added to the price of those articles, but there were numerous charges for freight that were small and difficult to apply directly; for instance, express and postage. All such expense was taken into the account for "Freight and Handling." Warehouse operating expenses were also charged to this account by closing the operating accounts monthly into G. L. 18. A percentage added to warehouse issues and credited to the freight and handling account, absorbed these expenses into feature cost in proportion to the value of materials used. The balance in the account as kept was a debit balance and not allowed to exceed a percent of the total value of material on hand in the warehouse equal to the percent added to warehouse issues. The rate applied to issues was varied from time to time as found necessary to keep the debit balance within reasonable limits.

No. 19 Goods in Transit. Many articles were shipped from distant points and purchased f. o. b. point of shipment. Invoices were often received before delivery of materials. The obligation to pay began with the shipment, so immediately upon receipt of

such invoices they were registered and charged to G. L. 19, credit being given to accounts payable. When the materials arrived and the corresponding invoices had been approved, the proper inventory or feature account was charged and G. L. 19 credited. For convenience all bills were run through this account. The balance was always a debit balance representing the value of invoices received but not yet approved for payment. This will be discussed more fully in the chapter on "Accounting for Materials and Supplies" (page 62).

Group III—Accounts Receivable.

No. 21 Personal Accounts Receivable. Amounts due the Department of Engineering and Construction for services rendered or for sales to persons or firms and for which cash payment was to be made, were debited to this account upon rendering the invoices. It was a control of the personal accounts receivable ledger, described on page 87, in which an account was kept for each individual and firm.

No. 22 Coupon Advance Account. Coupon books were sold to employees by field clerks and used as currency at the District stores. When a stock of books was given to a field clerk, a charge was made against the "Coupon Advance Account" and a credit to G. L. 45, "Unredeemed Coupon Books." A detail of account No. 22 was kept to show the account with each clerk. When books were sold to employees, the charge was against the employee and a credit to the proper field clerk's account in G. L. 22. When coupons were redeemed at the store, the debit was against G. L. 45 and the credit to store operations.

No. 23 Freight Advance. Sometimes the District paid freight on shipments which should have been paid by the shippers or was chargeable to the shippers. In such cases the expense was charged to controlling account G. L. 23. A ledger account subsidiary to G. L. 23 was kept to show amounts charged to each person or firm.

No. 24 Construction Advance. This account was created to record transactions by which the District advanced money for construction purposes; for example, it was necessary to build transmission lines to carry power to the dams. The power company did not care to finance the construction, so the District built the lines for the power company or advanced the money to cover actual cost, and charged the "Construction Advance" account. Reimbursement was made by deducting a certain percentage from the monthly power bill and applying the amounts deducted to the credit of this account until the total advance was repaid.

No. 25 Uncollected Interdepartmental Transfers. This was similar to ordinary accounts receivable except that it represented amounts due the Department of Engineering and Construction for materials or services furnished other departments. Such amounts were charged to this account until they had been accepted and the transfer of credit completed on the books.

Group IV.—Construction Cost.

No. 31 Feature Cost. This was a control of the cost ledger wherein was assembled the cost of physical features. On pages

20 to 34 details of the physical divisions and the account number scheme applied to them are described. The entire construction cost was ultimately assembled in this account.

No. 32 Undistributed Plant. This was a control of the plant cost ledger, detailed accounts for which are described on pages 26 to 28 under the title "Plant Accounts." The balance was always a debit balance, being the book value of equipment in use; that is, the amount of the investment in equipment and machinery in place, less the amount charged to feature cost (G. L. 31) as depreciation.

No. 33 Undistributed Operating or Clearing Accounts. This was a control of the class of accounts referred to on pages 28 to 32. A ledger showing the classification in detail as there given was maintained. In this ledger the debits and credits were cumulative, so that totals to date were always available as well as the undistributed balance for each detail. The sum of all these balances was equal to the balance in G. L. 33. A credit balance indicated an over-distribution to feature cost and a debit balance indicated that not enough had been charged to features to cover these operating expenses. Upon completion of the work, the balance was closed to feature cost.

No. 36 Salvage. As machinery became worn out, or as soon as it had served its purpose and was no longer needed on the work, it was transferred to the salvage division, where necessary repairs were made and it was held for disposition by sale or otherwise. The cost of repairs and selling was charged to salvage, detail of the account being explained as clearing account 066 described on page 31. This was closed monthly to G. L. 36. The balance was a debit balance representing the book value of machinery and equipment of all kinds not in use.

No. 37 Discount. This was a credit account representing cash discounts taken for prompt payment of bills. When an invoice was received, the gross amount was taken into cost or inventory at once. All discounts obtained were accumulated as a revenue which served to reduce the construction cost as a whole and were not credited to any particular feature. A subdivision of the account was kept in the ledger to show discounts on cement bills separately from those on all other bills.

No. 38 Miscellaneous Revenue. This was another credit account used to assemble revenues of a miscellaneous nature which could not be applied to any particular feature or item, and which served as a reduction of the total construction cost. Overhead charges on all work done by the District for outside parties went into this account. Interest charged by the District on overdue invoices rendered to outside parties was another item.

No. 39 Revenue from Freight Adjustments. A contract was made with a freight service bureau to check all freight bills with a view to locating overcharges by transportation companies. It was desired to know what the result of the check would be, and this account was arranged to take the credit for all freight refunds. The balance, a credit, was applied to the reduction of construction cost as a whole.

No. 40 Maintenance Cost. During the latter part of the construction period, some expense of a maintenance nature was incurred on completed units. This work included cutting weeds, reseeding banks, clearing drift, repairing roads, etc. It was done by the construction forces and the cost assembled with the construction cost. Account G. L. 40 was assigned as a control for this work to keep it separate from construction. It was transferred monthly to the treasurer's office where an adjustment was made by charging the maintenance fund and crediting the construction fund. Under classification of cost on page 32 will be found a detail of maintenance accounts.

Liability Accounts. Group V—Accounts Payable.

No. 41 Unpaid Labor. As soon as labor earnings were known for a pay period, which was weekly for the construction work, the total amount was charged to feature cost and credited to the account for "Unpaid Labor." There was usually a lapse of several days, and often longer, before payment was made and this often carried over the close of the fiscal period or end of the month. When payment was made, the charge was to G. L. 41 and credit to "Cash Disbursements." The balance was always a credit, representing amounts due for labor earnings.

No. 42 Unpaid Purchases. This account recorded the liability for purchases and invoices for miscellaneous services. When an invoice was received, it was registered and charged either to "Goods in Transit" (G. L. 19), direct to feature cost, or to some inventory account, and credited to G. L. 42. When payment was made, the charge was to G. L. 42 and the credit to cash disbursements. The balance was the outstanding liability for bills rendered against the District. For a full discussion of the method of handling invoices, see the discussion of the invoice, page 98, and payment of bills, page 96.

No. 43 Unpaid Earnings on Construction Contracts. This account was a control of the detail ledger showing amounts earned on formal contracts. Upon submission of estimates by engineers, showing contract earnings, the total was charged to feature cost and the amount due credited to G. L. 43. When paid G. L. 43 was debited and cash disbursements credited. The balance was the District's liability for contract work.

No. 44 Unpaid Contract Holdbacks. This account was closely related to G. L. 43. When contract estimates were rendered, the portion earned which was to be retained as a holdback until completion of the contract was credited to this account. When payment of a holdback was made, this account was debited and cash disbursements credited. Details showing the holdback on each contract were kept in a subsidiary ledger similar to that for G. L. 43.

No. 45 Unredeemed Coupon Books. When coupon books were issued to field clerks, G. L. 45 was credited. When the coupons were redeemed at the store, G. L. 45 was debited and the store credited. The balance represented the District's liability for the value of coupons outstanding. See explanation under account No. 22, "Coupon Advance Account" (page 37).

No. 48 Unpaid Interdepartmental Transfers. This was the reverse of account No. 25. It represented the charges against the Department of Engineering and Construction for materials furnished or services rendered by other departments pending the adjustment by transfer on the books of the treasurer's office.

No. 49 Railroad Adjustment. Railroad relocation played a very considerable part in the work of the District. The necessary construction was handled by procedure different from that employed for the other work. According to agreements made with the railroads affected, all work would have been done by the railroads or by contractors working under their supervision, and the District, upon presentation of bills covering the outlay, would have reimbursed them for all expense incurred. In practice it was found more advantageous for the District to let some of the contracts directly, or, in some instances, to do work with its own forces.

Whenever expense was incurred directly by the railroads, the details of accounting were carried on their books, payment being made to them by the District as suitable bills were presented. When the District arranged for work, either by contract or by force account, all accounting was carried on the District books.

Account No. 49 was a liability account to record the value of railroad materials furnished by the B. & O. Railroad Company in connection with the relocation of their tracks, but no bills were submitted and it was necessary to estimate the value. This was a temporary account and was closed upon adjustment of material bills with the railroad company.

Group VI—Reserve Accounts.

No. 47 Reserve for Accident Insurance, Taxes, etc. This account was subdivided to show reserves created for specific purposes, but was reported in one item on the balance sheet. The purpose of creating reserve accounts was to absorb in feature cost all expenses relating to current periods. Casualty insurance was carried with the State Industrial Commission, rates being fixed in advance for each six-month period. Based on this rate a percentage was added to labor distribution, charge being made to feature cost and credit given to G. L. 47. When the premium, based on the total of labor for the period, was paid, the amount of the premium was charged to G. L. 47 and credited to "Cash Disbursements." The rate was usually higher than the actual charge for insurance, so adjustments were made by crediting back the excess to features.

Group VII—Investment. This group was in effect the cash account. It was through these accounts that reconciliation was made with accounts in the treasurer's office. For example, disbursements by the treasurer on requests issued by the Department of Engineering and Construction were charged on the treasurer's books to the department and appeared as a credit to cash on the department books. Collections turned in to the treasurer by the department were credited to the department on the treasurer's books and appeared on the department books as a reduction in investment or debit to cash. Transfers received and transfers

issued on interdepartmental business were handled in the same way.

No. 61 Disbursements. Effort was made to maintain a cumulative total of disbursements and this total was carried in this account through the disbursement voucher register. The debit for vouchers paid was usually to accounts payable and credit was to this account (G. L. 61).

No. 62 Transfers Received. The only difference between this account and No. 61 was that it represented charges from other departments against the Department of Engineering and Construction, while No. 61 represented direct disbursements by the department. Entry came from the journal, debit being to G. L. 48, unpaid interdepartmental transfers, or if entry had not previously been made to accounts payable, direct to feature cost.

No. 71 Collections. For statistical purposes, it was desirable to carry all cash collected by the department into a separate account, rather than apply such revenues direct to cash disbursed. This was a debit to the cash account, the credit usually being to some account receivable. Credit was sometimes direct to feature or operation cost, in case the revenue had not been previously applied and charged to accounts receivable.

No. 72 Transfers Issued. Credits on the books of the treasurer to the Department of Engineering and Construction, for supplies furnished or services rendered to other departments, were recorded in this account as a debit. The only difference between this account and No. 71 was that in this case no cash was collected, the transaction being handled by transfers on the books of the treasurer.

The sum of accounts No. 61 and No. 62 less the sum of accounts No. 71 and No. 72 was the net investment by the department in flood control works. This was reconciled with the books of the treasurer every month, at the time of closing the department books. The "tie-in" with the treasurer's books was through these four accounts.

With this method of grouping general accounts, upon closing the accounts after all bills have been paid, accounts receivable collected, and disposition made of equipment and material, only two groups of accounts remain on the balance sheet, one for construction cost and one for investment. The net construction cost on the debit side equals the net investment on the credit side.

During the course of an audit of the District's accounts, criticism was made that the general accounts (treasurer's books) of the District did not show a complete record of assets and liabilities, in that the construction department was charged with disbursements only as made. To meet this situation and at the same time to avoid complicating or revising the accounting work of the District, three accounts were opened by the construction department to transfer to the treasurer's records all assets and liabilities shown on the construction books except cost and investment. The total of cash on hand, inventory accounts, and accounts receivable was transferred to the treasurer in one lump sum, credit being given to an account called "'Administrative

Adjustment, Assets" (G. L. 30). On the monthly statement this was shown as a credit (in red) on the asset side. The contra entry was a debit to a similar account designated "Administrative Adjustment (Contra)" (G. L. 80) carried on the liability side of the statement. The total of accounts payable and reserves was also transferred to the treasurer's books in a lump sum, debit being given to an account of the same nature designated "Administrative Adjustment, Liabilities" (G. L. 50), which was shown on the liability side and the contra or credit was placed to the same account as the contra for the transfer of assets, viz., G. L. 80. These adjustment entries were made as closing entries at the end of each month and were reversed immediately in opening accounts for the next month. No particular benefit was derived from this somewhat involved process, but it accomplished the desired result of showing the current assets and liabilities on the general books without complicating the books of the construction department.

Auditing. Section 57 of the Conservancy Law (104 Ohio Laws, pages 13 to 64 inclusive) provides that,

> "At least once a year, or oftener if the court shall so order, the board of directors shall make a report to the court of its proceedings and an accounting of receipts and disbursements to that date which shall be filed with the clerk of the court. Thereupon the court shall order the auditing of said accounts by public accountants of recognized standing who shall file their report thereon with the clerk of the court."

The audit went further than an analysis of "receipts and disbursements," as contemplated in the law. Checks were made of the warehouses and inventory of equipment and plant accounts. A complete financial statement showing assets and liabilities was submitted to the court.

On April 29, 1921 (Approved May 14, 1921), the General Assembly of the State of Ohio passed a law amending Section 57 of the Conservancy Law and placing the responsibility for the audit of the District's books upon the bureau of inspection and supervision of public offices. The portion of the amendment relating to the audit follows:

> "The bureau of inspection and supervision of public offices shall inspect and supervise the accounts and reports of the District and all laws pertaining to said bureau shall be applicable to such inspection and supervision. In no instance shall the books as kept to date of first audit be changed, provided however that the system of the keeping of the books now in vogue may be changed thereafter upon recommendation of the Auditor of State."

Under this law the forces of the State Auditor examined the accounts beginning with January 1, 1921. From that date the audit was made in detail, and in addition all receipts and disbursements from the beginning of the work were given a general examination. No changes were made in the system of accounting.

CHAPTER V

ACCOUNTING FOR LABOR

In the preceding chapters, the discussion has been confined to the general outline of the system and the relation of the various general accounts to each other in the system as a whole.

The chapters which follow will deal with details and methods, tracing the records from sources of original entry through to the final accounts.

Keeping Time. The timebook was the original source of entry for all personal services. The District used two forms of timebook; one for salaried employees who were paid semi-monthly, and one for labor paid on a daily or hourly basis, on which payment was made weekly. Figure 2 shows the form first referred to (Form E-8). It was arranged to record at the left of a double sheet the number, name, and position of each employee. Then follow sixteen small columns for the record of employment, headed from 1 to 15 and from 16 to 31. In using this form, time was inserted in hours, actual time worked being shown in large figures and allowance for overtime in small figures in the same block. To each name was assigned about five lines and the time employed on different kinds or classes of work was entered on separate lines. This book has the usual columns for total time, rate, amount earned, etc. The second book was arranged to automatically assemble cost by account numbers. (See Figure 2.) Any number of sheets necessary could be assigned for a given date. There are three general columns on each page, each column designed to record the time employed on a job or item of work. The timekeeper, in making his rounds, recorded each man by number, under a column assigned for the job on which the man was working. The other sub-columns are for time, rate, amount, etc. The total of each amount column was the amount earned chargeable to any account number for a given date. Of course, several columns were often required for one account. Time was recorded in the same manner in both books. The timekeeper usually made the rounds twice daily. To provide a check on the timekeeper's work and avoid mistakes and possible padding of rolls, a supplementary record of time was kept by the foreman in the form of a daily labor report. This was a card on which to indicate each man's number or name, hours worked, and on what work engaged. These time records were compared by the field clerk and differences adjusted before posting time to the payroll.

The above applies to timekeeping in the field. For office time, the semi-monthly book, but not the daily time card, was kept. In the machine and electrical shop a daily time ticket took the place of the foreman's daily report. This ticket was handed to

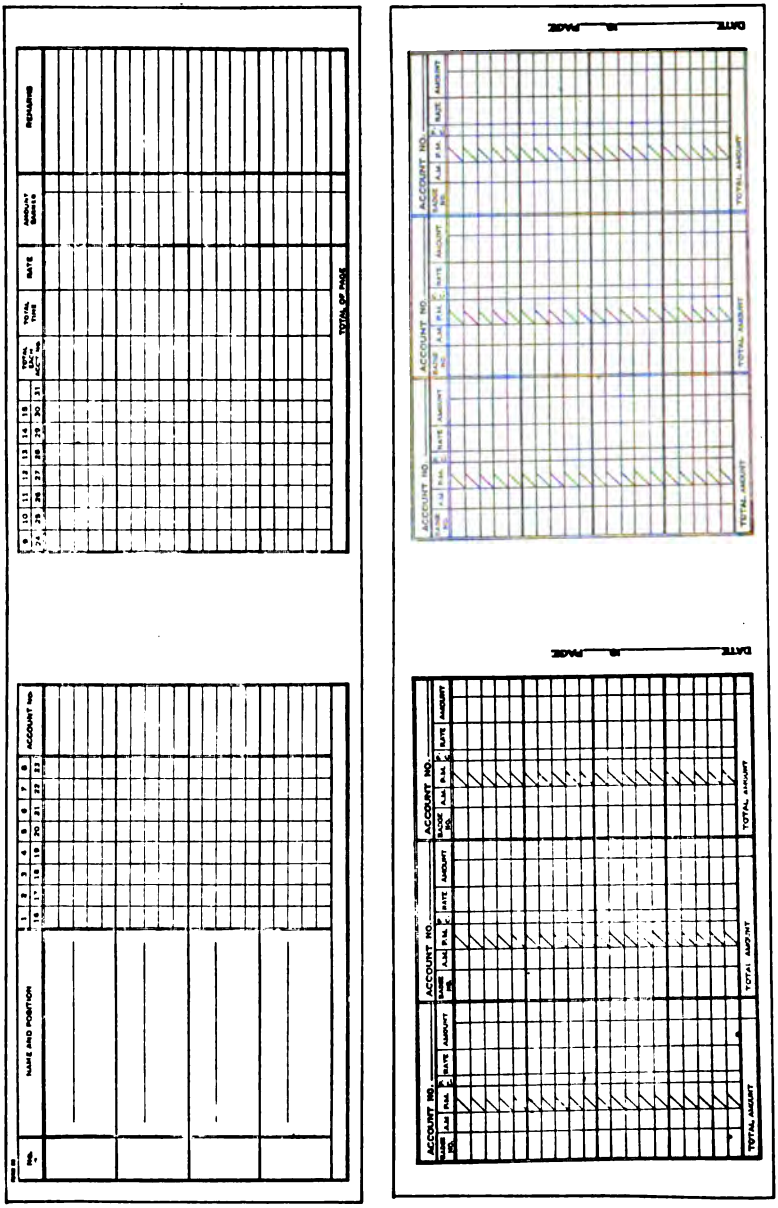


FIGURE 2—TIMEBOOKS—The upper is Form E-8 for salaried employees; the lower is the one for labor.

an employee when he began work in the morning and at noon and he deposited it at the shop office on leaving. The employee indicated on this ticket the time in hours he spent on each job or order. It was necessary that the total time on the ticket agree with the payroll time kept by the shop timekeeper.

Figure 3 shows the form of payroll used. It was posted daily by the field clerk or timekeeper, closed weekly, and forwarded to the paymaster. Deductions for meals, coupon books, and other charges to employees were shown on the roll as well as the amount due. The payroll was kept in duplicate by carbon process so that the field clerk could retain one as a field record. Accompanying each payroll was an abstract and distribution sheet (Figure 4) which gave a summary of all transactions affecting the roll and the labor account. This was prepared by the field clerk, one copy going to the accounting division to become the original source of entry for labor. One copy was kept by the field clerk. A copy of the abstract also went to the auditor and a fourth copy was filed with the payroll. The total of the labor distribution sheet was required to equal the amount earned as shown on the time book. The time book was filed with the distribution sheet to support the entries made by the accounting division.

Before proceeding further on the subject of accounting for labor, a brief description will be given of the routine of placing men on the roll, of transfers, changes in rate, discharges, etc. A series of brass checks or badges was provided for each job or feature for which a separate payroll was provided. These were numbered, a certain series being assigned each feature. They bore the number of the feature as well as a number for the employee. Before a man started work, he reported at the field clerk's office, where he was assigned a number and given his check, which served thereafter for identification. The field clerk made out a notice of employment (Form E-18) showing the feature number, badge number, the man's name and address, rate time of beginning work, and other personal information. When this had been approved by the superintendent, the name was recorded on the payroll and the notice forwarded to the paymaster. Thereafter, the name appeared on every payroll, whether the man worked or not, until notice had been given by the field clerk, on Form E-21 "Payment in full," that services had been discontinued. This form showed in addition to name, feature, number, etc., the reasons why services were terminated, earnings since last pay period, deductions, and net amount due. It was usually carried to the paymaster's office by the employee. The new payroll each week was prepared by the paymaster's office, in duplicate, by the use of an addressograph machine, from information contained on the previous roll, the employment notices, and pay-off slips. This was forwarded to the field clerk and the only names to be added were those of men employed since the beginning of the week.

The discharge slip (Form E-22), a little ticket printed in red, was given to discharged employees by the superintendent. This

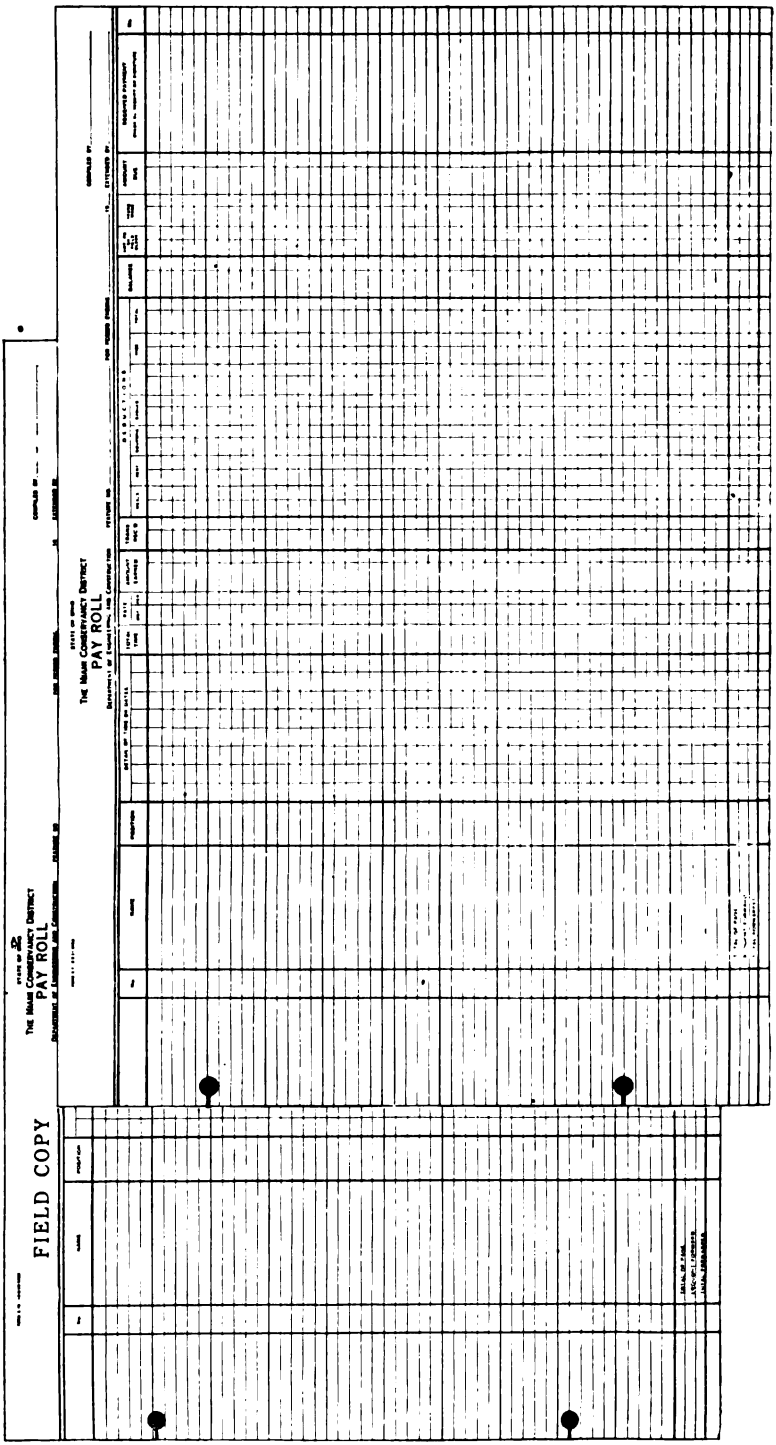


FIGURE 3—PAYROLL USED FOR EMPLOYEES PAID WEEKLY

was presented by the employee to the timekeeper, who made appropriate entries in his time records and issued the pay-off slip.

Form E-20 "Transfer" was used to transfer employees from one feature or payroll to another. It required the name, badge number, feature from and to which transferred, date, time, kind of work, amount earned and due up to date of transfer. The record was made on the transferring feature and after signature by superintendent, field clerk, and timekeeper, the slip was forwarded to the receiving feature where a new badge number was assigned and the slip forwarded to the paymaster as notice for change on the payroll.

Whenever a change in rate was made, Form E-19 was prepared showing in addition to name, badge number, feature, etc., the reason for the change, the old and new rates, time of taking effect, etc. After signature by superintendent, timekeeper, and field clerk, it was forwarded to the paymaster for his record and thereafter the employee appeared on the payroll at the new rate.

Paying Labor. Executive employees who were paid an annual salary were put on what was known as the treasurer's payroll, the distribution of which was given to the accounting division in lump sum. Checks were issued by the treasurer direct. Other permanent employees were put on what was called the headquarters payroll and a separate distribution sheet and abstract was made by the headquarters time clerk. The latter roll was handled in all other respects in the same manner as the labor roll except that payment was made by check.

Upon completion of the field payroll, the field clerk made out a card (Form E-15) called a wage receipt for each employee. It showed the name and number of the employee, the date, time, amount earned, deductions, amount due, and contained a place for the signature of the employee. This card was handed to the employee who held it until the paymaster's visit when, after affixing his signature, he exchanged it for his pay envelope. By having this notice in advance of payment, the employee had an opportunity to check his time and any adjustment of errors could be made before the paymaster's visit. Upon receipt of payrolls from the field, the paymaster checked all computations, rates, etc., with his record and drew a check for the entire payroll, obtaining cash which was distributed in envelopes containing the amounts due each employee. The paymaster was directly responsible to the treasurer's office and his cash account was entirely independent of the department accounts. The department was charged with the net amount of the payrolls through the disbursement voucher register. For work located near Dayton, pay envelopes were prepared in the Dayton office of the paymaster, but for outlying features arrangements were made to secure funds from local banks, and pay envelopes were filled at the bank. Different pay dates were fixed for the different features. Payment was made at the feature office, the employee presenting himself at the window with his badge and receipt card. He showed his badge for identification and exchanged his card for the pay envelope. The field clerk was usually present

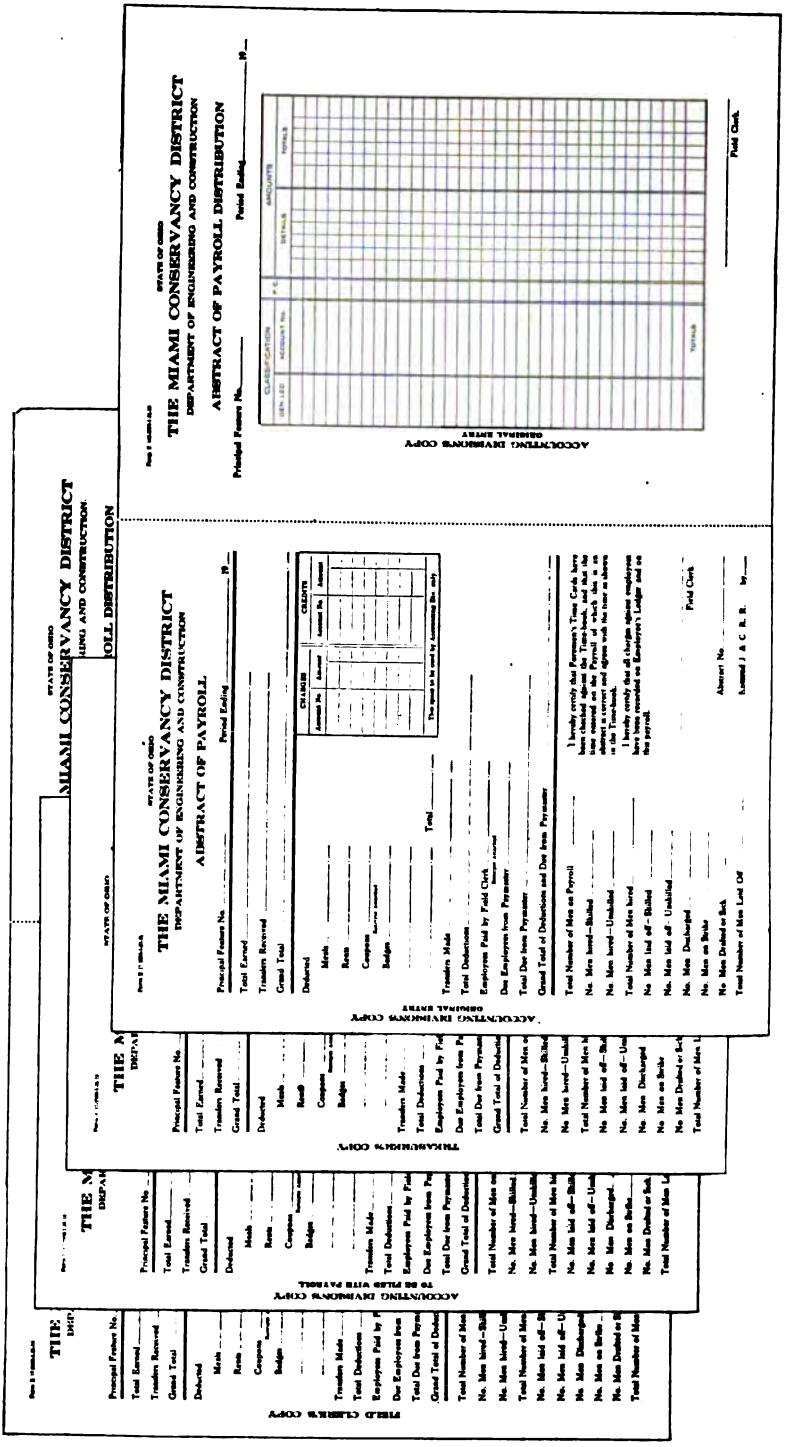


FIGURE 4—ABSTRACT OF PAYROLL AND OF PAYROLL DISTRIBUTION

to help identify men and prevent irregular practice on the part of laborers. The pay period ended on Tuesday and all payrolls were paid by the end of the week.

At the beginning of the work the pay period was semi-monthly for all employees. This was later changed to four periods a month, and finally to weekly periods. The latter agreed with the general practice in the community and during the war period it was difficult to hold labor on any longer basis.

Quitters and discharged men who demanded payment immediately were paid by the field clerk. Each field clerk was under bond and was given advances by the treasurer for this purpose. Upon advancing money to pay labor, the field clerk took up the

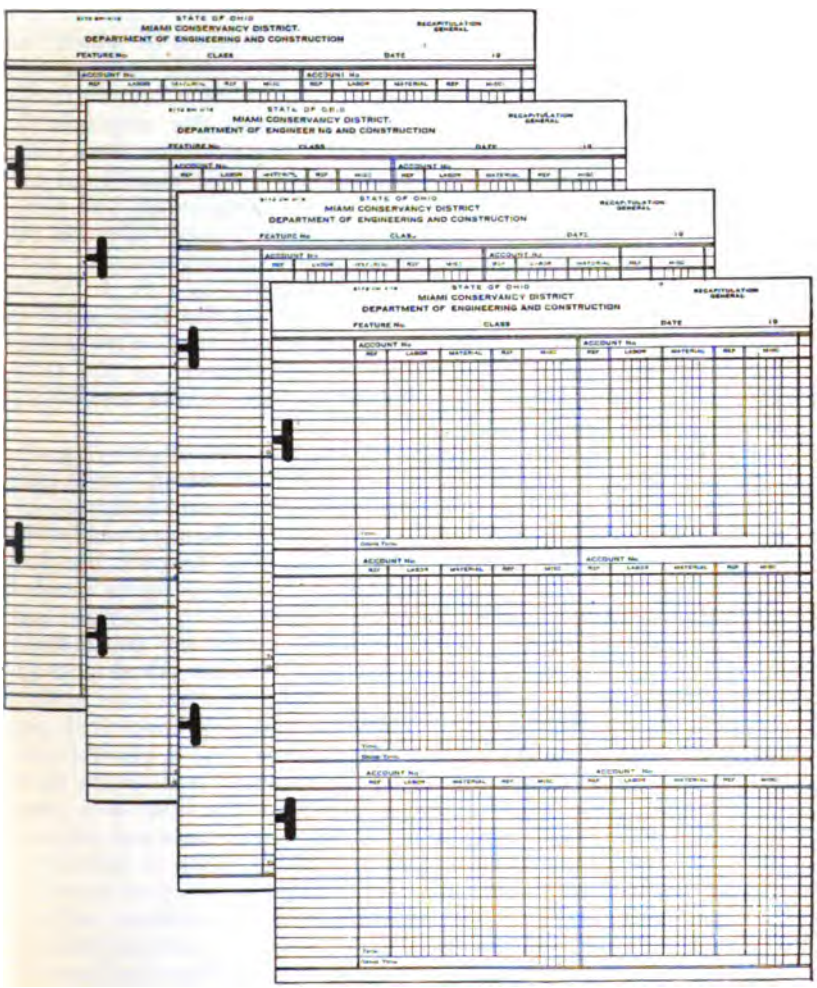


FIGURE 5—RECAPITULATION SHEETS, FORM E-172

wage receipt, made the entry in the advance column of the payroll, and was reimbursed for such advances by the paymaster.

Bookkeeping. The assembling of labor cost in the headquarters office began with the labor abstract and distribution sheet (Figure 4). These sheets showed the total labor earnings, deductions, etc., which agreed with the amounts on the timebook and payroll. They were forwarded with or immediately following the payroll. The totals on the abstract affecting each controlling account were entered in the journal. Total earnings were debited to feature items, deductions were credited to the various operations for which the deductions were made and the net amount due was credited to unpaid labor (G. L. 41). The distribution sheet was posted by the cost clerk to recapitulation sheets (Form E-172, Figure 5) on which were columns for each account number and sub-columns for labor, material, and miscellaneous. There was also a reference column in which was posted reference to original source of entry. The journal was posted to the general ledger by use of general classification sheets (Form E-171, Figure 6) which summarized the amounts for each controlling account and from which the postings were made in the general ledger proper. At the close of the month the columns of the recapitulation or detail sheets were footed and totals carried to the appropriate detail cost ledger. As explained in the chapter on cost classification, there were three classes of cost: G. L. 31 for feature cost, G. L. 32 for plant, and G. L. 33 for operation accounts. Recapitulation sheets for the three controls were the same except for color, and the detail sheets were assembled in three books for convenience in posting to the three ledgers. Before posting to the ledger each book was totaled and balanced with the control.

For convenience of bookkeeping, the expense of carrying casualty insurance was added, on the distribution sheet, to the total earnings for labor. This was a certain rate per one hundred dollars for different classes of employees, the rate being fixed by the State Industrial Commission. The rate being known, the field clerk simply added the amount to the charge for each account on his distribution sheet and the amount went into feature cost along with the regular labor earnings. Credit for the amount added went to the reserve account (G. L. 47) described in Chapter IV (page 40).

Vacation and Sick Leave. Vacation and sick leave with pay was granted to certain classes of employees. As a general rule, this privilege was granted only to the salaried class or to those who were not allowed pay for overtime. For the first three years of the District's operations, individual vacation time was allowed to accumulate. This was changed in January, 1921, so that thereafter all vacation with pay had to be taken in the year in which it accrued. One day a month was allowed for vacation and two days a month for sick leave. As the work on certain features drew near completion, it was realized that some leave had accrued which had not yet become a part of the cost, and to avoid belated charges to cost after features had been reported as completed, a

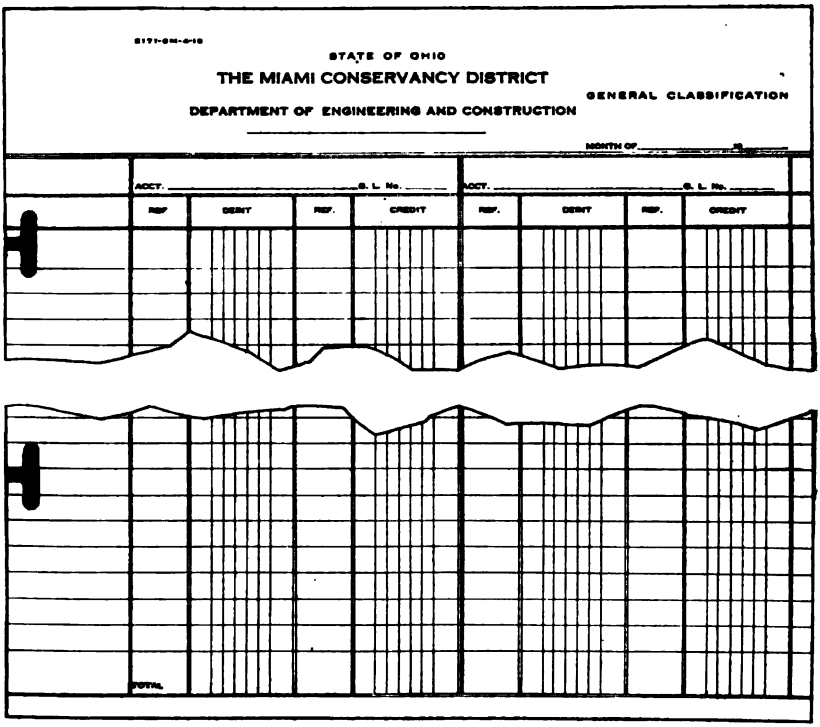

STATE OF OHIO
THE MIAMI CONSERVANCY DISTRICT
GENERAL CLASSIFICATION
DEPARTMENT OF ENGINEERING AND CONSTRUCTION
MONTH OF
ACCT. G. L. No.
REF. DEBIT REF. CREDIT REF. DEBIT REF. CREDIT
TOTAL

FIGURE 6—GENERAL CLASSIFICATION SHEETS, FORM E-171

plan was adopted of adding a certain percentage to labor cost for this purpose and crediting the addition to the reserve account. Then, when vacation time was paid, this reserve was charged. The amount to be added was estimated at three percent for field forces and five percent for office forces to cover both vacation and sick leave. For convenience this was added to the rate charged for casualty insurance and both were carried into the cost with one operation. A subdivision of "Reserve Account" G. L. 47 was kept to indicate how much had accumulated for each purpose.

CHAPTER VI

ACCOUNTING FOR MATERIALS AND SUPPLIES

The accounting for materials and supplies properly begins with their purchase. However, it is not the intention to discuss the technique of purchasing here. It will be sufficient for an understanding of the accounting and business methods of the District to describe briefly the purchasing organization, records and forms, and their relation to the accounting for the District.

Purchasing and Purchase Records. Purchasing was in charge of a separate division located in the headquarters office and reporting direct to the chief engineer. The purchasing agent had nominal jurisdiction over warehouses to the extent of responsibility for materials to be carried in stock. In theory, all purchasing for the District was done by the purchasing agent, but it often happened that division engineers and field clerks, acting under the general supervision of the purchasing agent, bought articles of small value which were not in stock in the warehouse. The garage superintendent issued most of the orders for car repairs and supplies for the garage. The steward did considerable purchasing for the messes and stores, buying especially foodstuffs of a perishable nature.

For the larger purchases, bids were secured after advertising, which was sometimes informal by circular or even by telephone. The question of advertising and the methods of securing bids were left largely to the judgment of the purchasing agent.

For construction materials and machinery, specifications were prepared by the engineers and construction manager, but for miscellaneous purchases and material for stock the specifications were usually prepared by the purchasing division from information secured from the master mechanic, superintendents, and others.

The purchasing office was organized into sections, each of which handled certain classes of purchases. The larger and more important articles were handled by the purchasing agent personally. One man was responsible for foodstuffs and supplies for sale in stores, another handled repair parts for machinery, etc.

Theoretically, all articles purchased should go through the general warehouse and from there be issued to the feature. In practice, bulky articles and carload materials were shipped direct to the work where they were to be used, and often charged to the item of work direct without going through the warehouse accounts. This short-cut saved considerable expense in the case of articles that could be handled in this manner without confusion. In most instances the records were put through the warehouse. Each division office was supplied with a stock of warehouse orders (Form E-9) and demand was made on the general warehouse by the use of these orders. In the case of telephone and verbal orders, confirmation on the regular form was insisted on as a matter of record. If the articles wanted were in stock, immediate delivery

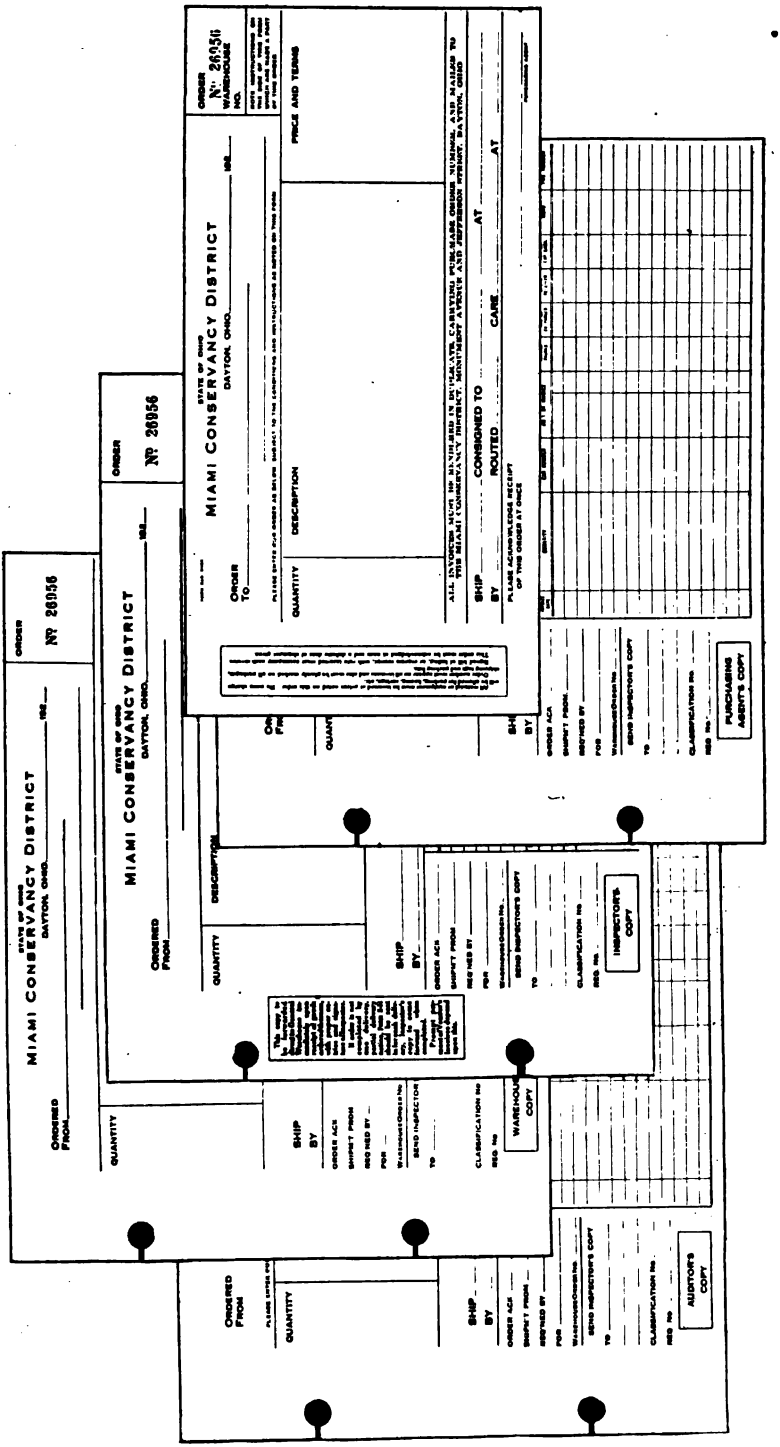


FIGURE 7—GENERAL PURCHASE ORDER, FORM E-23
The various copies were differentiated by color.

was made from the warehouse, but if not, the warehouse superintendent made out a purchase requisition on the purchasing agent. No standard form was used for advertising, which, as stated, was often done by letter or verbal inquiry. Figure 7 is a copy of the general purchase order. The first copy went to the vendor, one copy remained in the purchasing agent's files, one copy, known as the "Inspector's Copy" was forwarded to the receiving office or person, and one copy went to the auditor. Upon receipt of the material, the receiving office listed articles received on the inspector's copy, signed and forwarded it to the purchasing agent, where it was compared with the invoice and used as a basis for approving the payment. With this system it was not necessary to send the invoice to the receiving feature for O. K., but usually a copy of the invoice was sent for the information of the receiving office. Every order was numbered in advance and the number served as the basis of the purchasing agent's file. A cross reference to the warehouse order number and the purchase requisition number was made on the purchasing agent's copy, so that the file could be readily located if any one of the numbers was known.

A more informal order form was used for purchases made in the field. This was called an "Emergency Purchase Order" (Form E-205) and a copy is shown as Figure 8. The original of this order went to the vendor, one copy was retained by the purchaser, one copy was sent immediately upon issue to the purchasing agent for his approval, and one was retained until receipt of the goods and was used as the inspector's copy as outlined above. In this case, the invoice was sent to the Dayton office along with the inspector's copy with recommendation for payment. It was required that every invoice received be tied to some order by number, thus giving an excellent check against duplicate payments.

A weak point that develops in any plan of allowing field purchases, is that it is almost impossible to get field employees to issue orders in advance. They will order by telephone or verbally and later confirm by written order if they do not forget it. In some cases they do forget it and the order is written upon receipt of the bill, for record purposes only. But, on the whole, the use of the emergency purchase system is economical because it permits the securing of minor and unusual articles at less expense and in less time than if it is rigidly required that all materials be secured by a central purchasing department. It also reduces to a minimum the necessity for field employees to advance their own funds for such purposes. In construction work it is impossible to foresee every little requirement and order in advance. Unforeseen conditions which require changes in methods and plans are encountered. Most unexpected breaks occur in machinery. Emergency purchases are more common than in a concern where the work is routine. Any saving in price is incomparable to the loss which might be sustained by forcing crews and equipment to wait for minor supplies or parts. System is necessary in buying, but it must be so arranged that these

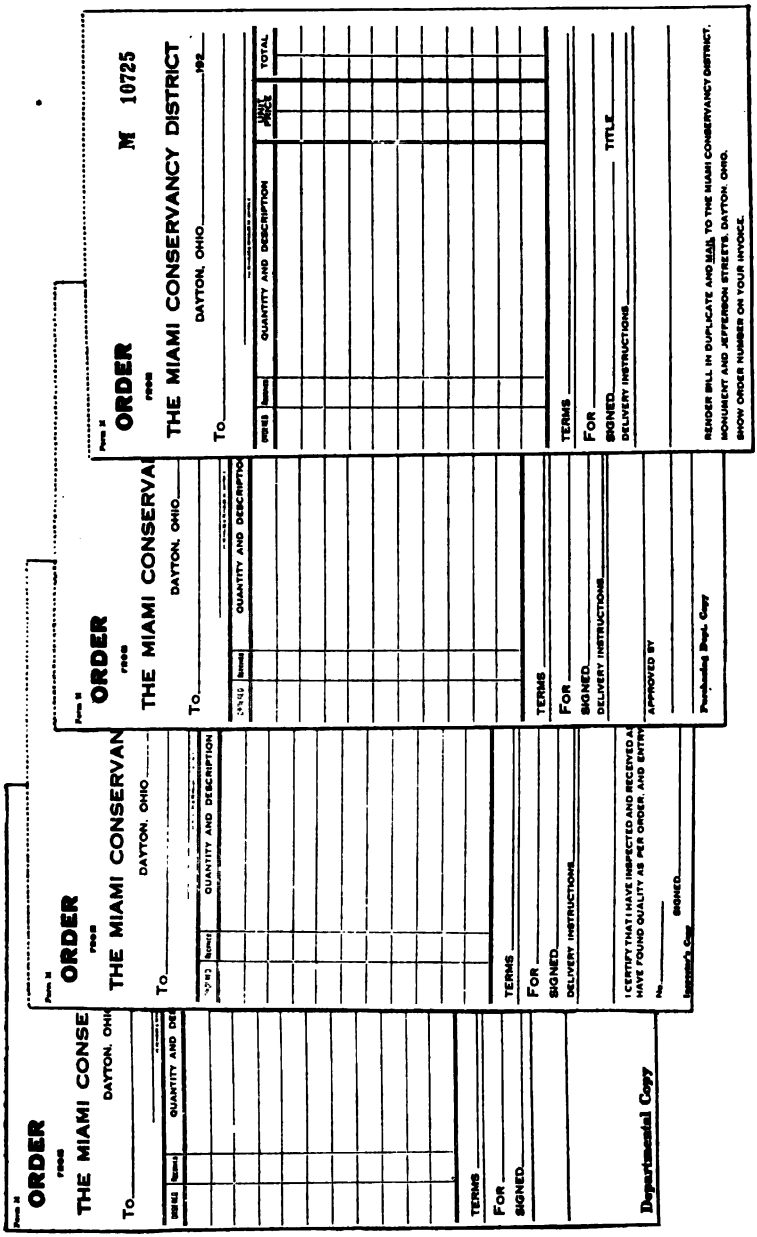


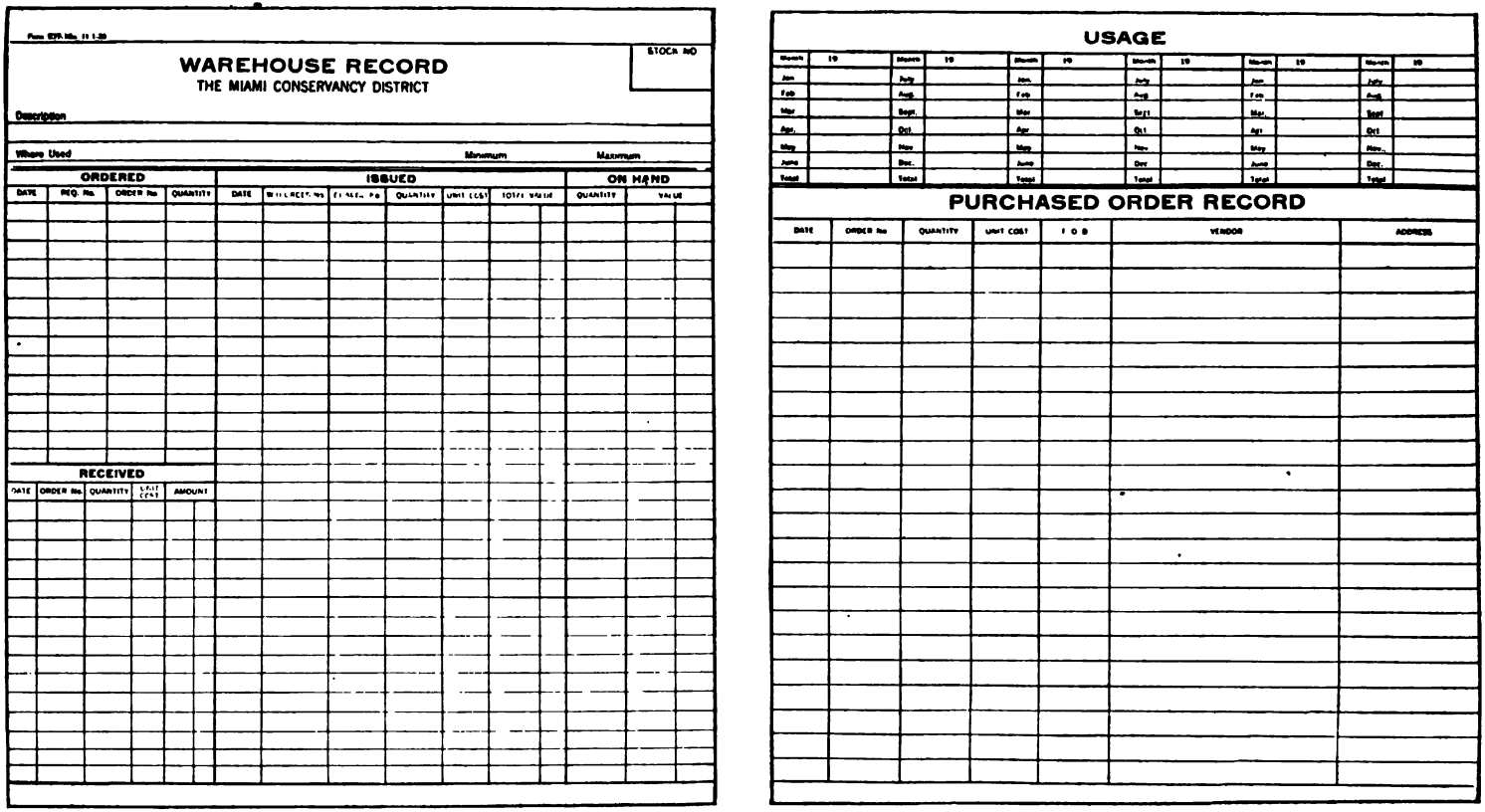
FIGURE 8—EMERGENCY PURCHASE ORDER, FORM E-205
The various copies were differentiated by color.

emergencies can be met with the least possible delay. Too strict adherence to regulations may easily result in innumerable delays and expense. There is also another phase of the matter. Construction men are busy getting the job done, and in the rush of work often fail to anticipate what afterwards seem to be obvious requirements, and a heavy burden is put on a purchasing department if it is required to meet all conditions imposed by the work and at the same time make all purchases at lowest possible prices.

The purchasing division's file of orders may be roughly divided into two classes: open and closed. Unfilled orders were filed in post binders, a separate binder being used for different classes of purchases, and in some instances where a great many orders were given to a single firm a binder or portion of a binder was assigned to a firm. Geographical classification was also used, the orders placed with Dayton firms being kept separate from orders placed at distant points. The "follow-up" system consisted in merely leafing over the unfilled order binders. When an order was filled, the record was completed by transferring to the purchasing agent's copy the information shown on the inspector's copy and the order was then taken out of the open file and placed in a vertical filing case by firm name, all papers relating to a given order being placed in the same folder. The inspector's copy with the certificate of delivery thereon, went with the invoice when forwarded for payment and was ultimately filed with the invoice and voucher in the accounting division.

Transportation. Freight is a very considerable part of material expense on construction work, and the adoption of a competent traffic plan is important. In the District's organization, transportation matters were handled by an expert traffic man in the purchasing division. Routing of shipments was made by him and he approved all bills for freight and express before payment.

The Warehouse System. The general warehouse was located in Dayton and there was a field warehouse at each of the larger features except the Dayton local improvement which ordered direct on the general warehouse. Materials issued from the main warehouse to features went through the local warehouse records although they did not physically become a part of the local warehouse stock in a great many instances. Control accounts were kept in the accounting division for each warehouse, but details were shown on stock ledger cards kept in the warehouses. At the beginning of the work, a stock ledger card was made out for each individual purchase of any article. Under the original plan these were to be made out by the purchasing division and forwarded to the warehouse with a copy of the invoice or order, but the scheme failed to work satisfactorily and it was changed, a card being adopted similar to the standard stock ledger sheet. Figure 9 (Form E-77) illustrates the card used by the general warehouse and Figure 10 (Form E-89) is the field warehouse card. They are essentially the same, the difference being mainly in size and arrangement of columns. In the heading, the usual space is provided for articles, description, etc. Columns are pro-



WAREHOUSE RECORD
THE MIAMI CONSERVANCY DISTRICT

STOCK NO

Description

Where Used Minimum Maximum

ORDERED				ISSUED							ON HAND	
DATE	REQ. No.	ORDER No.	QUANTITY	DATE	WITHDRAWN No.	CHARGE No.	QUANTITY	UNIT COST	TOTAL VALUE		QUANTITY	VALUE

RECEIVED				
DATE	ORDER No.	QUANTITY	UNIT COST	AMOUNT

USAGE

Month	19	Month	19	Month	19	Month	19	Month	19	Month	19
Jan		July		Jan.		July		Jan.		July	
Feb		Aug.		Feb		Aug		Feb		Aug.	
Mar		Sept.		Mar		Sept		Mar.		Sept	
Apr.		Oct.		Apr		Oct		Apr		Oct	
May		Nov		May		Nov.		May		Nov.	
June		Dec.		June		Dec		June		Dec.	
Total		Total		Total		Total		Total		Total	

PURCHASED ORDER RECORD

DATE	ORDER No	QUANTITY	UNIT COST	F O B	VENDOR	ADDRESS

FIGURE 9—WAREHOUSE RECORD CARD, FORM E-77
This was used in the general warehouse only. The view on the left is the front; the one on the right is the back.

vided to show amounts ordered, received, and issued. On both cards space is provided for money accountability as well as item accountability. The total of card balances at any one time was supposed to equal the balance charged against the warehouse on the books of the accounting division. During the first two years of the work, great difficulty was experienced in the warehouse accounting, but gradually the system was worked out on a practicable basis and very satisfactory results were obtained by the system here described. The variation between the warehouse man's record and the control became very small. This card record was in effect a perpetual inventory. Receipts and issues were posted to the card and balances carried forward promptly. The warehouse force was continually checking card balances with stock on hand, and periodical test checks were made by a warehouse inspector from the accounting division. Of course, differences were unavoidable, especially in bulk stock such as coal, lumber, iron and steel, oil, etc., where it was not feasible to measure or weigh each issue accurately. But, on the whole, these differences were surprisingly small in comparison with the volume of business handled. As soon as any variation was detected, adjustment was made. The aim was always to make the book balance of materials and supplies on hand reflect actual conditions as nearly as possible, and to enable the responsible employees to know the current stocks by reference to the card balances. Cards were also of value as price records and their careful maintenance made a purchasing agent's price book unnecessary. For some classes of material it was impracticable to post the cards at the time of making an issue. Bulky material was unloaded as near as possible to the place of consumption and necessarily much of the record of use came from reports of foremen, supplemented by test checks of balance on hand from time to time. Night foremen usually carried a key to the warehouse and did not always report even the taking of articles from the warehouse. Therefore, it cannot be claimed that the warehouse accounting, as kept by the District, furnished an adequate system upon which to base personal accountability for District property. The expenditure necessary to provide such a system would have been out of proportion to the benefits derived and would have unduly hampered the work of construction. The system provided, however, a basis of obtaining costs approximately correct and the moral effect of even nominally holding a warehouse man responsible for the use of material was conducive to economy. On construction work of this sort, a compromise is necessary between accurate accountability and economy of operation.

In addition to the stock ledger card, the field warehouse records consisted of a receiving blotter (Figure 12, Form No. 180) and issuing blotter (Figure 13, Form E-181). Everything received on the job was listed on the receiving blotter whether it went into warehouse stock or not. If it went direct to the work, an entry was immediately made on the issuing blotter and a record was not necessary on the warehouse card, but even in such cases it was found convenient to make a card for future

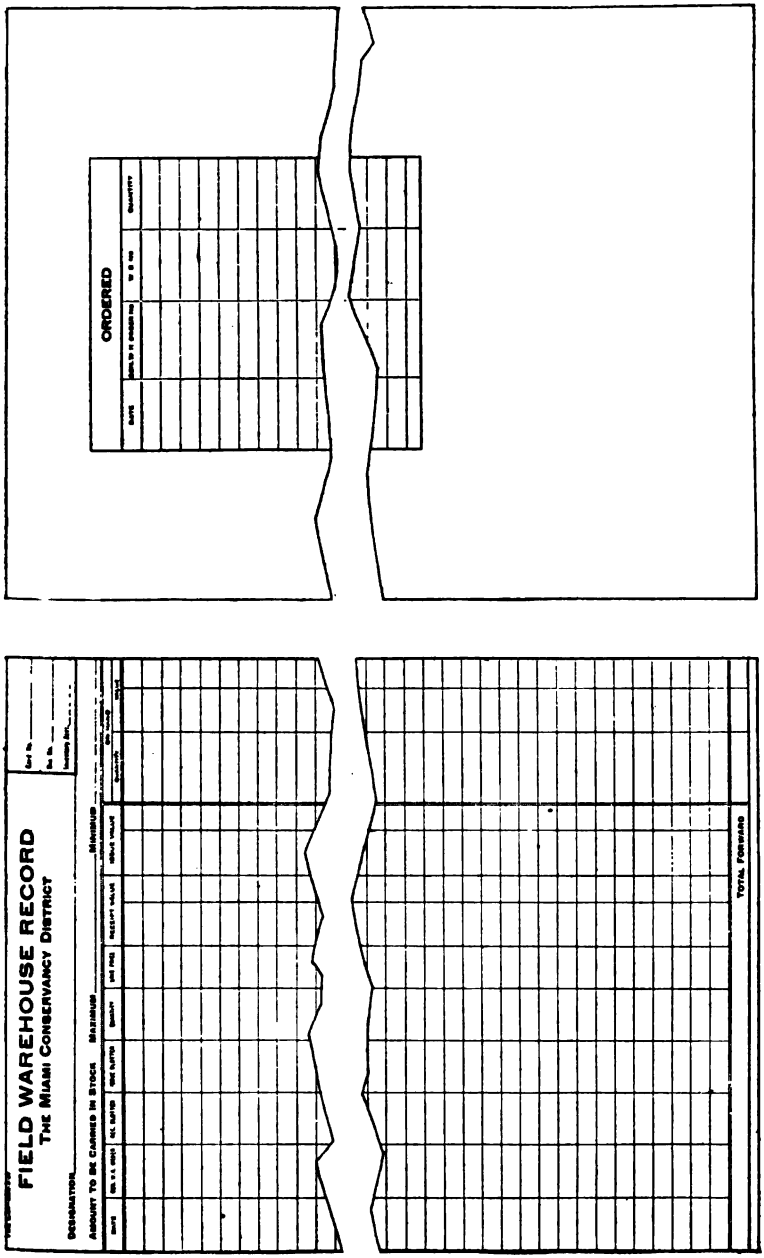


FIGURE 10—WAREHOUSE RECORD CARD, FORM E-89

This was used in the field warehouses only. The view on the left is the front; the one on the right is the back.

reference. The cards, being alphabetically arranged, served as an excellent index when information was wanted as to prices and the movement of materials. It was through the blotters that charges and credits were made to local warehouses and they served as original sources of entry for the accounting division. They were made in duplicate, the first copy being sent to the accounting division daily, and the carbon retained for field record. They were self-balancing, having both debit and credit columns.

Upon receipt of a shipment, articles were listed on the receiving blotter and the amount entered in the debit column as a charge against the inventory account (G. L. 16).

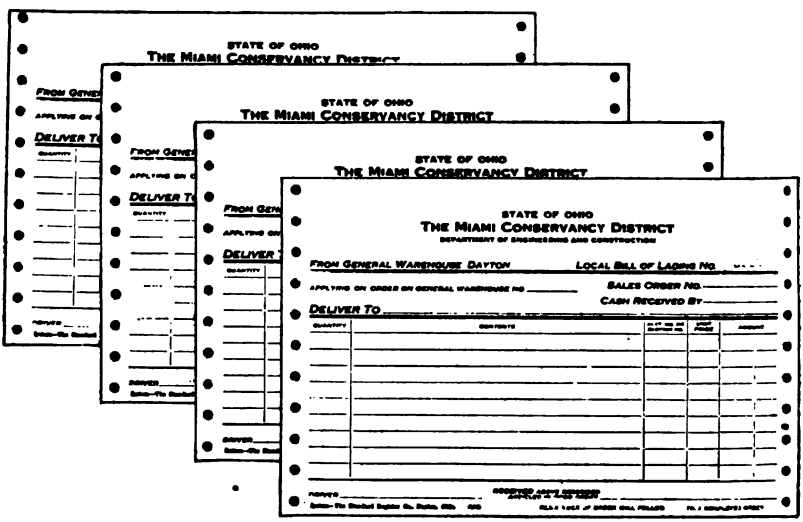


FIGURE 11—BILL OF LADING
The various copies were differentiated by color.

The credit entry was in the appropriate column for "Direct Purchases" (G. L. 19), "General Warehouse," "Transfer," or "Return from Work," depending upon the source from which goods were received. Reference columns were provided to indicate the document from which entry was made.

When material was issued to the work, it was listed on the issue blotter. The debit entry was in the column provided therefor, a sub-column indicating the account number to be charged. The credit entry was in the column for the proper inventory account. Issue blotters were signed by the person taking or authorizing the issue and were approved by the superintendent.

When these blotters were received in the accounting division, entries were made from the column totals (except in the cases where sundry accounts were involved, when a recap was made showing the amount affecting each account) to the materials or miscellaneous column on the recapitulation sheets described on

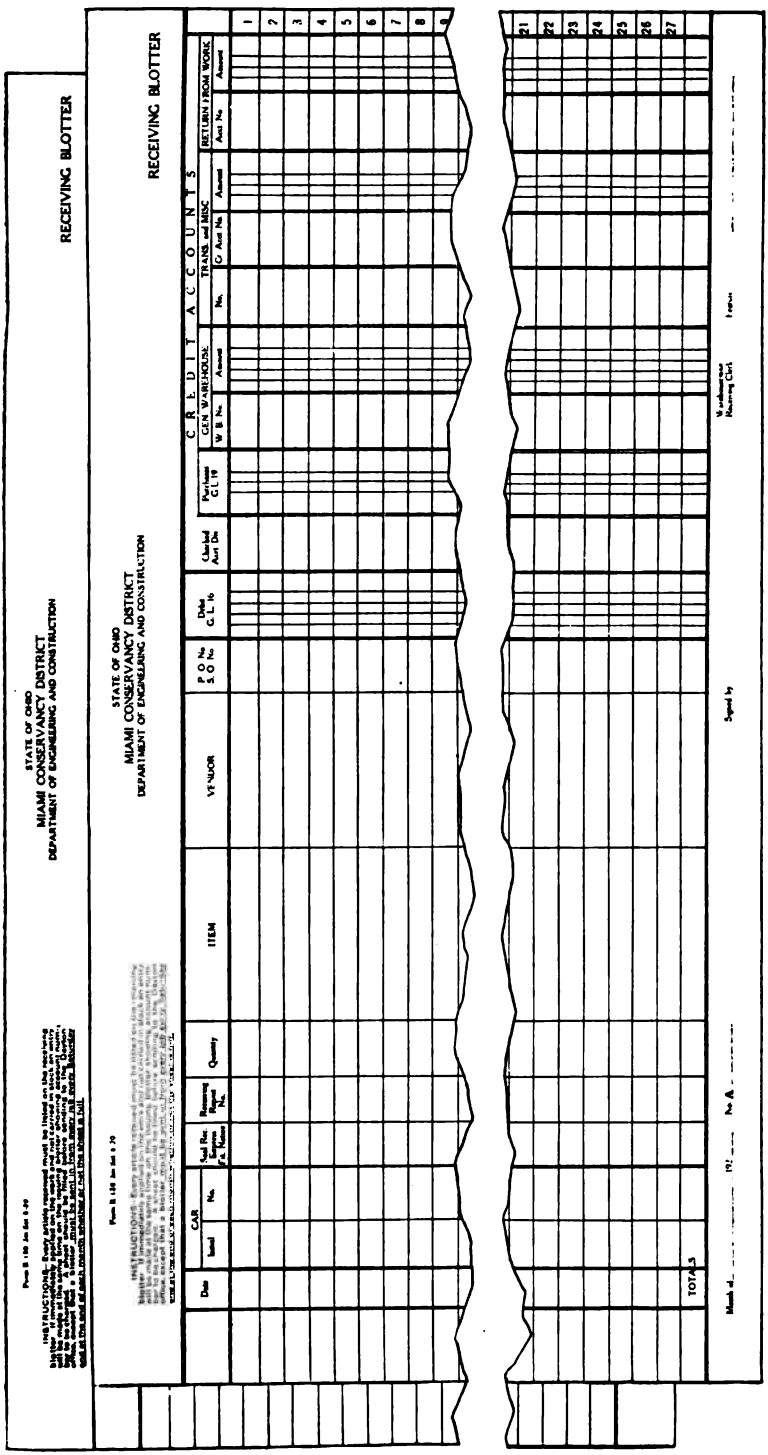


FIGURE 12—RECEIVING BLOTTER, FORM E-180

This was in general use in all warehouses. The two copies were differentiated by color.

page 50. In this way the movement of material and supplies was recorded and assembled for posting to the ledger accounts the same as for labor, and when the columns on the recapitulation sheets were footed and totaled, only one entry to the ledger was necessary for the entire month's business.

For the general warehouse at Dayton, receipts were handled in the same manner. It will be noted that there is a column on the field receiving blotter for credit to the general warehouse and the general warehouse received no credit until receipt was acknowledged by an entry on these blotters. Issues from the general warehouse were recorded on bills used in the Standard Register. (Figure 11.) These were written in quadruplicate, the original and one copy being sent to the receiving job as an invoice, the third copy being retained by the warehouse, and the fourth copy going daily, on the register roll, direct to the accounting division. The fourth copy served as a tickler to check job receiving blotters. It was required that all of them be accounted for and if issues were made to features on which there was no warehouse, and hence no receiving blotter, charge was made with credit to the warehouse account on a special voucher prepared in the accounting division. The originals or white copies of the bills, after being signed by the receiving feature, were forwarded to the accounting division where they were filed numerically to support the entries on the various receiving blotters and special vouchers referred to. No changes were allowed on either the bills or blotters after they were sent in, errors being corrected by adjustment entries on the blotters or by correction bills with reference to the original entries by number.

In the early stages of the work, a great deal of difficulty was experienced in getting prices by the time shipments were received. Often materials were received and used before invoices were received. This was particularly true of repair parts on which prices could not be obtained in advance. To overcome this difficulty, an estimated price was put on the warehouse copy of the purchase order, and if the actual price was delayed, this estimated figure was used for the initial entry. Invoices, when passed for payment, were checked from the receiving blotters in the accounting division and adjusted through the use of adjustment account G. L. 15, described on page 35. If the differences were large, adjustment was made to the accounts affected by the original entries.

Direct Charges. Theoretically, all charges to features for purchases should have gone through the warehouse accounts, but as a matter of fact there were many items which it was inconvenient to so handle. Some of the field clerks' purchases were examples. Such charges were made direct to items of work from the first entry in the invoice register or from the disbursement voucher register. Charges were assembled on the recapitulation sheets referred to on page 50 from all books of original entry.

Goods in Transit. Reference was made in Chapter IV, on page 36, to general ledger account No. 19 which carried the amounts of invoices received pending approval. Some such ac-

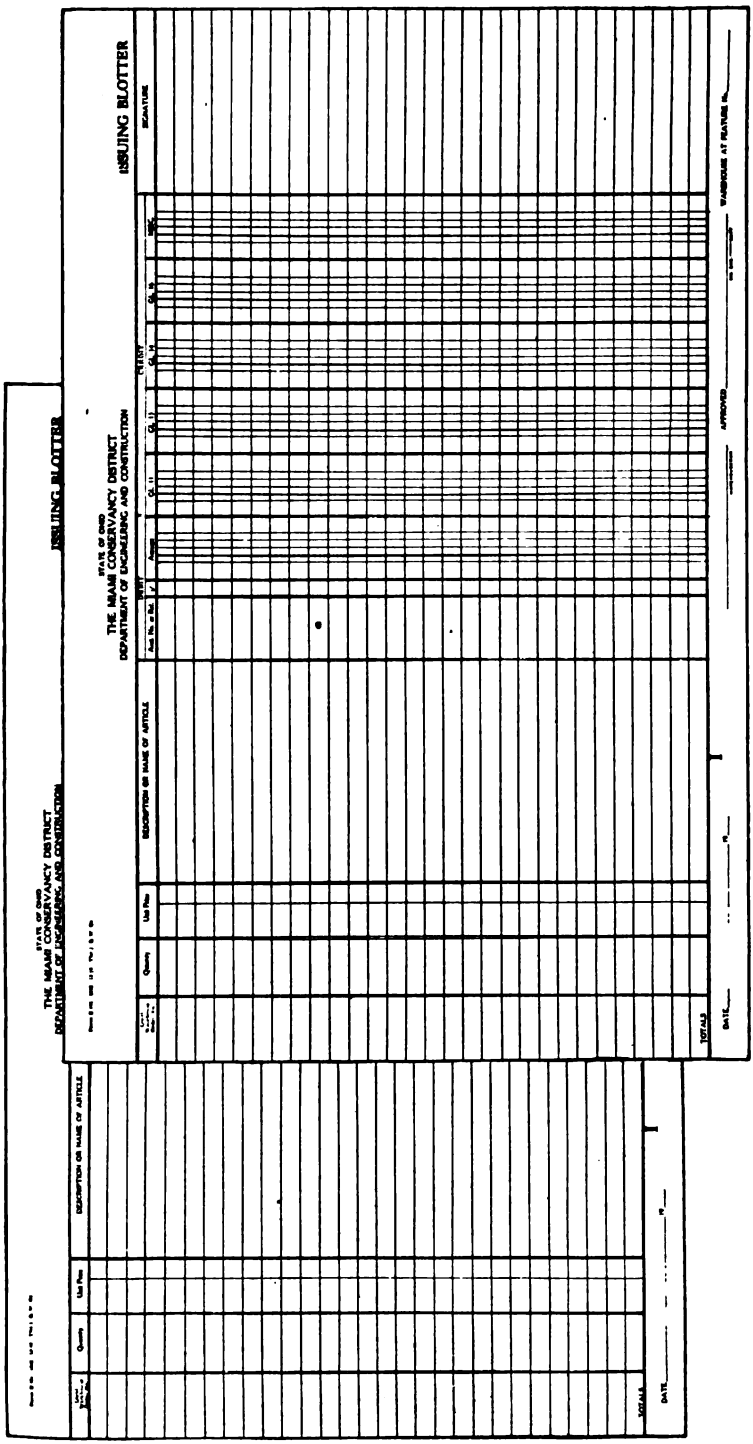


FIGURE 13—ISSUING BLOTTER, FORM E-181

This was in general use in all warehouses except the general warehouse. The two copies were differentiated by color.

count was necessary if the liability for purchases was to be taken up immediately upon receipt of invoice and if a proper record of invoices received was to be kept. The debit to G. L. 19 came from the invoice register and was made at the time the invoice was recorded and credited to accounts payable. G. L. 19 was cleared by the credit shown in the G. L. 19 column on the various receiving blotters. If the entries on the receiving blotters had been in all cases for the same amounts that were taken up on the invoice register, the clearing of G. L. 19 would have been a simple matter, but occasionally corrections were made or invoices lost. This involved numerous adjustments. Sometimes invoices were changed and then charged direct, leaving an erroneous balance in G. L. 19 and corresponding credit in accounts payable. The objection to an account of this nature is this difficulty of keeping it under control. (See discussion of invoice register, page 87.)

CHAPTER VII

PLANT AND EQUIPMENT

Definition of plant as used in accounting for the Miami Conservancy District is given on page 26 in the discussion of classification of cost for plant. The purpose of using plant accounts rather than to charge the cost of machinery and its installation direct to features is twofold:

1. To distribute the investment in plant over the entire period of construction in the cost records.
2. To enable a proper proportion of the investment in plant to be charged to different features using the same equipment.

To these there might be added the advantage of accumulating cost of equipment and machinery and its erection in a separate group of accounts for statistical purposes.

The distinction between **plant** and **equipment** should be made clear before discussing the accounts for this class of expense. Any expense for labor, material, supplies and equipment involved in the building of a plant used in the production of some other structure or product, is a proper charge to plant. Equipment is composed of the non-expendable articles of machinery, forming parts of a plant, which have a value separate and apart from the plant and are capable of being transferred from plant to plant or used independently. A screening and washing outfit for sand and gravel is a good example. As a working unit capable of turning out washed sand and gravel, it is a true plant. The motors, crushers, screens, etc., installed in the plant, are items of equipment. For convenience in accounting, however, the term plant has been applied also to the aggregate cost of small tools or utensils, assembled in one account, later to be charged off to features gradually as depreciation. For example, the plant account numbered 06, "Miscellaneous Tools and Equipment," referred to on page 27 is a plant of this nature.

The first difficulty met in accounting for equipment is to determine where to draw the line between expendable and non-expendable articles. The latter term describes equipment in a general way, but there are numerous articles embraced within the term that are so small in value or whose life is so short that it is impracticable to class them as equipment. In the classification adopted by the District, an arbitrary value basis was used. Any article costing less than $100 was considered outside the class of equipment, regardless of its period of usefulness. This was done to facilitate accounting, as it was not desired to encumber the equipment records with the thousands of small articles which were difficult to trace or identify and the cost of which did not unduly affect the current unit cost of items of work. This arbitrary division is not altogether satisfactory. It does save accounting, but many articles such as, for example, office equip-

ment, are buried in cost and yet have a very considerable sale value. In the writer's opinion, a good rule to follow is to charge to equipment all articles which can be identified by number or otherwise and which have a life of not less than six months under ordinary wear and which can be transferred from one installation to another without losing their identity. Small tools may be carried in a tool account, but it is impracticable to account for them individually under the usual conditions surrounding construction work.

The system used for marking District equipment was a combination of letters and numbers. Articles in each class or kind of equipment were numbered in series beginning with one. This number was preceded by another one, arbitrarily assigned, indicating the class or kind, and also by the initial letter of the equipment name for further ready identification. Thus D-16 indicated dragline excavators. The complete number for machine No. 1 was therefore D16-1. Twenty was the number for motors, so motor No. 10 carried the number M20-10. Sometimes these numbers were put on metal tags and attached to the machines; sometimes the numbers were stamped on with steel dies; and whenever practicable, the number was painted on the machine in some conspicuous place.

During the period of organization and assembling and installation of plant, equipment went on the jobs so rapidly that it got ahead of the accounting. It was not until the late summer of 1918 that an adequate record of equipment on the job was obtained and an accounting system worked out for it. At that time the numbering was completed and the office record brought up to date. It was always difficult to keep it up to date. Maintaining the record was a job which was not compulsory and it happened that the equipment man was occasionally transferred to work supposedly more important, with the result that the equipment record suffered.

The equipment record kept by the accounting division centered about the equipment card (Figure 14, Form E-14). It contained a space in the heading for name of article, number, manufacturer, date of purchase and description. Then followed the record of use, in the form of transfers, appraisals, charges for depreciation, and final disposition. When equipment was received, four copies of the card were prepared. One was retained in the accounting division, one was furnished to the equipment man, one was given to the chief engineer, and one was supposed to follow the equipment as a field record. It did not always follow. The accounting division record was kept up to date in a fairly satisfactory manner, but the transfers from job to job were so frequent, and it was so difficult to get the construction forces to report transfers, that the record frequently became incomplete.

All equipment was charged to some appropriate plant account, classification for which is given in Chapter III, and the cards were filed by plants. A cross index, alphabetically arranged, was maintained, so that cards could be readily found. In addition to the cards, the office engineer kept in his files a record of

the larger items of machinery, with complete specifications and drawings. These were necessary in connection with the purchase of repair parts, replacements, etc.

As previously explained, the plant accounts contained not only the cost of the machinery and freight to site of work, but also cost of installation, dismantling, and major repairs in a great many instances. In fixing rates for depreciation all these expenses had to be taken into consideration. It was estimated that for equipment two percent per month of the first cost plus freight would be a suitable rental to charge each job on which it was

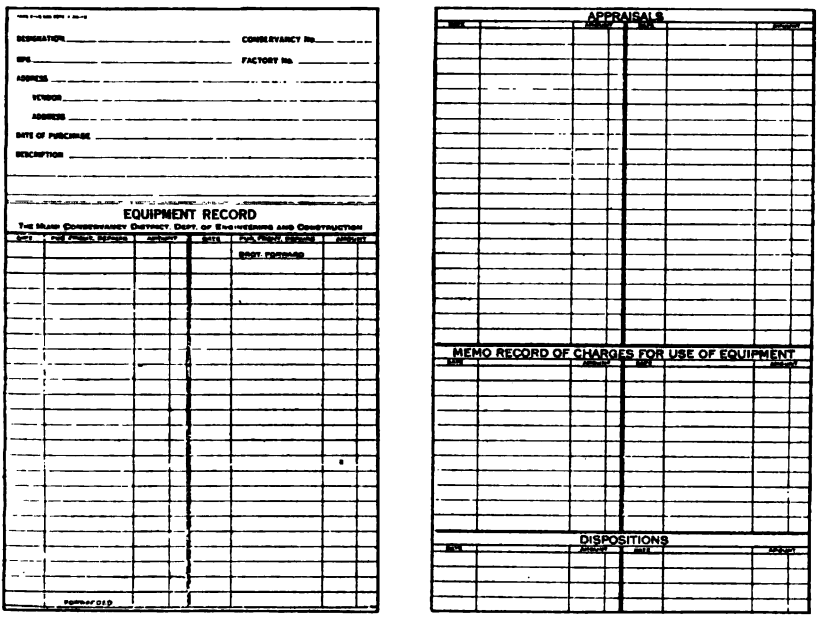


FIGURE 14—EQUIPMENT RECORD CARD
Front and back of card are shown.

used. The rate took into consideration the period of construction and the probable salvage value of the equipment upon completion of the work. No attempt was made in the beginning to fix different rates for different classes of equipment, but as time went on the rate was changed where it was found that more, or less, ought to be charged off as the work proceeded. In most instances two percent was found to be very satisfactory. For miscellaneous plant costs, a flat rate per month, varying to conform to the actual cost of installation and dismantling, was charged. An estimate of the probable life of the plant was, of course, necessary. In the case of motor vehicles two percent per month proved to be too much and the rate was lowered as the cars grew older and repairs became more extensive. All minor or current repairs were charged to operation, so that with this sliding scale of depreciation, the charge for use of equipment was fairly well equalized.

Transfers between jobs were made on a bargain and sale basis, much as though the deal were between independent business concerns. That is, the superintendent receiving the equipment was supposed to join with the superintendent transferring it in fixing a value. As a matter of practice, however, movement of equipment was usually ordered by the construction manager or master mechanic and neither they nor the superintendents were very much concerned about the records. The division engineer made out transfers (Figure 15, Form E-167) in quadruplicate, sent the original and one copy to the receiving feature after placing thereon his estimate of the value, kept one copy for his file, and sent the fourth copy to the accounting division, where it was used as a tickler to follow the transaction and as a means of keeping the equipment record. The receiving engineer, if he agreed on the price, signed the original copy of the transfer and forwarded it to the accounting division for entry. In case the two could not agree on a value, the matter was subject to arbitration. Usually the chief engineer or some one in his office fixed the price. Credit was then given to issuing plant and debit to receiving plant account.

The accounting division sent out depreciation lists every month charging the features at the prescribed rate for the use of all equipment shown charged to their plant accounts. These were checked in the field offices, account numbers against which to distribute the charges placed thereon, and the lists returned to the accounting division for entry of depreciation charges. Depreciation was charged as long as equipment was in possession of a feature unless notification was given of its availability for transfer. This requirement and the monthly lists which were forwarded to the job offices helped in keeping the record up to date.

Everything which left a feature, for which credit was to be given, was listed on the issuing blotter. When the transfer was made out, it was recorded on the blotter, credit being taken for the price named. The accounting division could not charge the receiving feature until the transfer was recorded on the receiving blotter for that feature. The debit was made, therefore, to a suspense account (065 C, referred to on page 31) until such time as it was taken up. The entries were as follows:

From the issuing blotter
Dr. Suspense (065C)
Cr. Transferring feature plant.
From the receiving blotter
Dr. Receiving feature plant
Cr. Suspense (065C)

The balance in 065C represented outstanding transfers and furnished an additional check on the movement of equipment. This worked both ways. If the transferring feature failed to record the transaction and the receiving feature did record it, the balance in 065C would be a credit. The objection to such an account was the detail work necessary to keep it under control, but it was worth while in order to get a check on movement of equipment and to properly record the cost. Numerous adjust-

ments, of course, were necessary because of differences in prices. It might be mentioned here that this system of transfers applied to materials and supplies as well as equipment, the only difference being in the accounts affected.

Salvage. Toward the close of the work the question of salvage became an important one. A salvage division was organized whose duty it was to put machinery in the best possible condition for sale. An account was opened to keep a record of the cost of this work. (See classification on page 31.) When a feature was completed and its equipment not required at any other feature, the equipment was transferred to the salvage division at a price agreeed on by the chief of that division and the transferring division engineer, credit for the agreed amount being given to feature plant. When all equipment used on a feature had been transferred to salvage, or otherwise disposed of, the feature plant was closed, the balance, either debit or credit, was absorbed in feature cost and a final cost report made. The disposal of equipment by the salvage division in no way affected this final cost report, even though the equipment might not be sold for several years after completion of the work, for it was estimated that on an average the sale price would just about clear the selling and repair expense. Equipment was supposed to be put in fair working condition before leaving the feature, but there was always something to be done to put it in presentable condition for sale. The excess in sale price over allowance made to feature plants went to cover this expense. For this reason the entire cost of the salvage division was closed into a general ledger account which represented the value of equipment not in use (G. L. 36).

It was not necessary that all sales of equipment go through

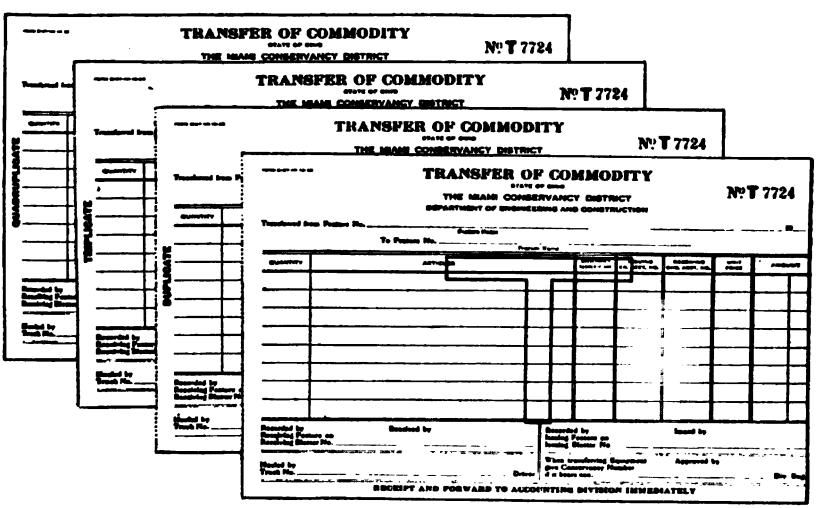


FIGURE 15—TRANSFER OF COMMODITY
The various copies were differentiated by color.

the salvage account. Many instances occurred where sale was made direct from the feature. In such cases, however, ten percent of the selling price was credited to the salvage account (066Y) to help pay the cost of general selling, all advertising and other selling expense being charged to the salvage operation account.

Upon completion of the railroad relocation, a large amount of track material was left on hand. The salvaging of this material was handled separately from equipment salvage. Most of the old track material was sold back to the railroad companies whose lines were relocated, and the proceeds credited direct to relocation cost. This credit was shown in a sub-account (Z) in the classification of railroad cost. (See page 24.)

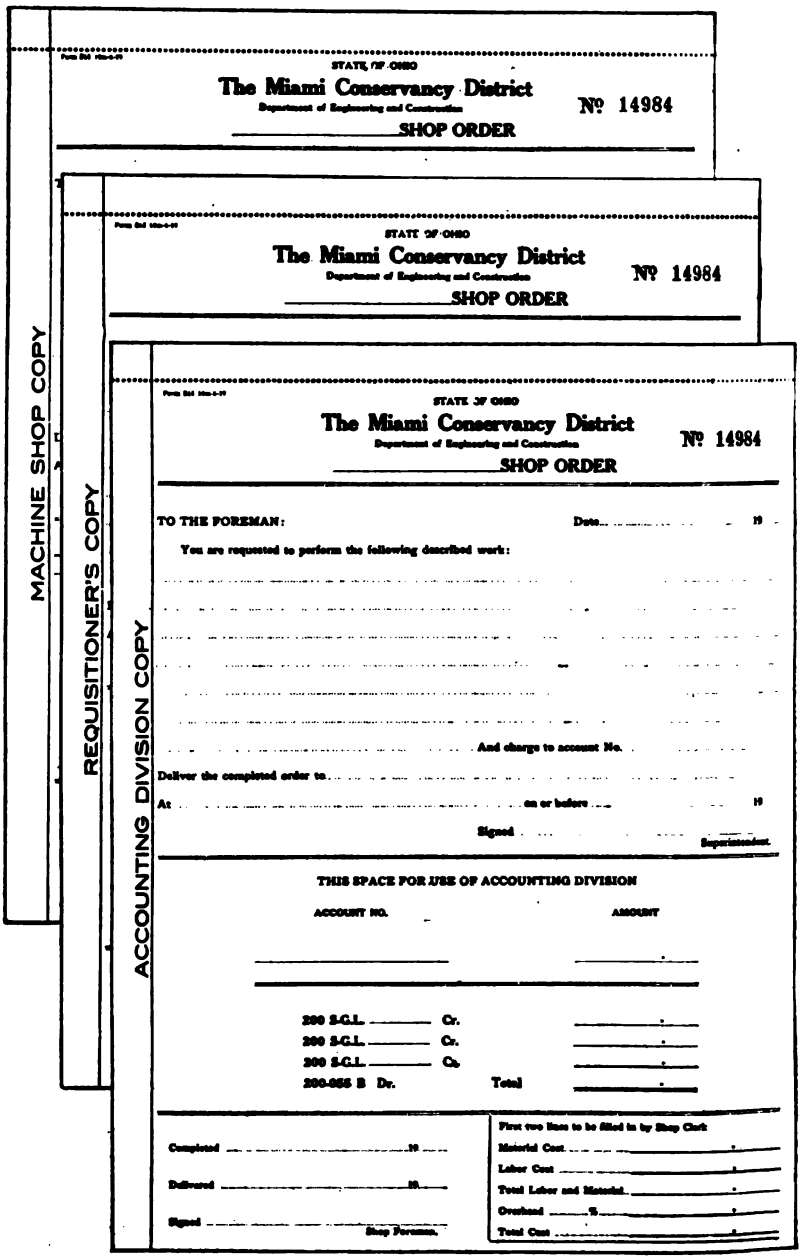
CHAPTER VIII

INCIDENTAL OPERATIONS

Mention was made in Chapter III (pages 28 to 32) of accounts for operations which were incidental to construction work, but which took charges of a deferred nature. All such charges must ultimately be absorbed in feature cost, except, of course, those operated for revenue only. These accounts were similar to accounts for production factors or centers in factory accounting and were kept for the same purpose: viz., assembling the cost of some special unit or factor for statistical purposes and later distribution, on some equitable basis, to jobs or processes for which the factor was operated. The more important of these operations will now be discussed in more detail.

Shops. On each feature there was a small shop, where a variety of work was done, such as machine work, blacksmithing, carpenter work, etc. The expense of this work was charged direct to feature cost items when conditions made such action practicable, but much of the expense of these shops was assembled in shop cost (Acct. No. 055) and cleared each month by distribution to the work for which the shops were operated. Transactions in the field shops were not large, as most of the repair work was done by the general machine and electrical shops in Dayton. Both of these general shops were operated on a job order basis. Orders for work were supposed to originate in writing with the superintendent or construction manager, but in practice they were usually sent in by telephone, and from various sources. A piece to be repaired came into the shop and the shop clerk wrote out the order himself in accordance with instructions given him. There was never much time for planning, or for making drawings or bills of materials. Most repair work was in the nature of a rush job and the clerical work in the shop was much more than it would have been under a system requiring advance estimates and carefully worked out bills of material. The very nature of the work made it impracticable to have such a system, however, as almost everything ordered done was for an emergency and the job that was most urgent had precedence regardless of the date of order.

When the shop clerk wrote an order, it was numbered and one copy handed to the foreman who proceeded to do the work. The form of order is shown in Figure 16 (Form E-66). All labor performed under authority of an order was charged to that order by using the appropriate number on the employees' time tickets. For some time a separate stock room or warehouse account was established for the shop, and orders signed by the foreman were made on this stock room for materials and supplies. These order tickets, when filled, went back to the shop clerk. He asembled all material and labor cost by shop orders, added a percentage to cover shop overhead, and forwarded the order to


STATE OF OHIO
The Miami Conservancy District
Department of Engineering and Construction
Nº 14984
SHOP ORDER

MACHINE SHOP COPY

REQUISITIONER'S COPY

ACCOUNTING DIVISION COPY

TO THE FOREMAN: Date 19

You are requested to perform the following described work:

And charge to account No.

Deliver the completed order to

At ..... on or before 19

Signed ..... Superintendent

THIS SPACE FOR USE OF ACCOUNTING DIVISION

ACCOUNT NO. AMOUNT

200 S.G.L. Cr.
200 S.G.L. Cr.
200 S.G.L. Cr.
200-055 B Dr. Total

Completed 19

Delivered 19

Signed ..... Shop Foreman.

First two lines to be filled in by Shop Clerk

Material Cost

Labor Cost

Total Labor and Material

Overhead %

Total Cost

FIGURE 16—SHOP ORDER, FORM E-65.

The first two copies were on bond paper. The last, or machine shop copy, was on yellow tag board. On the back was space for entering the time of the workmen. The copies were differentiated by color.

the accounting division where credit was given the shop account (055Y) and debit made against the feature ordering the work done. A copy of the order was sent to the feature when the charge was made.

The special warehouse account for the shop stock was discontinued in 1921. The stock was greatly reduced and the materials not in constant demand went back to the general warehouse, which was located close to the shop. All current supplies were then charged to shop material and material lists were written by the foreman as progress was made on an order. These lists took the place of order slips, and while possibly not so accurate as the regular order, the change was justified by a saving of one clerk in the shop.

On the books kept in the accounting division, all shop expense was charged to the shop accounts (055 or 056 A to X). Orders completed were credited to the shop credit account (055Y and 056Y). Both debits and credits were cumulative so that at any time a reference to the cost ledger showed the total cost of any class of expense, the total credit, and the balance. This balance approximated the value of material and labor on uncompleted orders, plus materials in the stock room.

Nothing was added to the shop orders for profit, the aim being to distribute actual operating cost to the features as accurately as possible. Variations in the balance occurred, and the rate of overhead had to be changed occasionally in order to absorb all overhead charges, the rate ranging from 15 to 33⅓ percent of the cost of labor and materials. In the distribution of shop overhead to orders, a flat rate was used. This, of course, was not absolutely fair to all orders. Those jobs which required much labor were charged more overhead than those on which the work was done largely by machines, but as the District was not in business for profit, absolutely accurate cost figures were not essential, and the additional expense of keeping a more elaborate cost system in the shops would hardly have been justified. Machine hour rates were used in connection with the operation of the locomotive crane at the shop, and with the information now at hand, rates could be fixed for use of other production units which might have resulted in more accurate cost distribution. Much difficulty was encountered in getting labor charged correctly. Men shifted from job to job frequently, and with no time-recording device in use, the actual time on each order was largely an estimate. The foreman and shop clerk tried to check the time distribution daily, but with the force working under emergency pressure nearly all the time, it was not carefully done, and consequently the costs were not always dependable in the early part of the job. Gradually these irregularities were eliminated and standards fixed so that during the last two years shop costs showed fairly accurate results.

Motor Vehicles. During the period of maximum activity on construction, the District had from sixty to sixty-eight motor vehicles in operation. Six of these were five-ton trucks, seven were two-ton trucks, thirty-one were light trucks and delivery

cars of various kinds, and from eighteen to twenty-four were passenger cars. The central garage, located at Dayton, was in charge of a garage superintendent with a force of mechanics and drivers. Most of the cars were operated out of Dayton, but some were assigned to features and were kept in the field. All repairs of any consequence were made at the central garage.

No attempt was made to assemble the cost of operating each car in the official cost records kept by the accounting division. The cost of operating the garage as a whole was kept as shown in the classification on page 29. At the garage there was a cost clerk who kept cost, by cars, on a large sheet containing columns headed to conform with the standard classification. He secured his information for labor from time tickets, and for materials and parts taken from the stock room, reference was made to the issue blotter. Overhead was distributed on the basis of labor expended on each car. Distribution of the garage expense was accomplished in two ways:

(1) A flat rate per day or hour was charged for the service of cars working out of the garage.

(2) A monthly rental was charged for cars assigned specially to feature offices. The rates, at first, were an estimate, but later were based upon actual costs as shown on the car sheets, and on total charges on the general books, against garage operations. These rates varied from time to time. The aim was to charge out actual cost only, so the rate had to be high enough at all seasons to care for periods of idleness when some of the cars were not in demand. In the latter part of 1921, the cost by cars was discontinued, as it was expensive to maintain and it was thought that sufficient information was at hand upon which to base future rates.

A separate warehouse account was kept for the garage, with perpetual inventory cards and other records the same as for the general warehouse. Most of the purchasing for this warehouse was done direct without using the general warehouse. Gasoline was bought in large quantities from tank wagons and charged to cars as filled. A daily sheet with car numbers printed thereon was used for gasoline and oil charges. This was turned in each night and one entry for the total made on the issue blotter. The garage clerk kept a record of services rendered on daily sheets from which statements were prepared. While a cost clerk was employed, statements were sent direct to features where they were approved, charged to some account, and forwarded to the accounting division for entry as a debit to feature cost and credit to garage. This plan worked very well. When the force was reduced, the daily sheets were sent direct to the accounting division by the garage clerk, charges were made on the books at once and notices sent to the features charged. This was quicker and cheaper but the charges were not so accurately made and some adjustments were necessary after field engineers received the notices.

Depreciation was charged to operation cost at the rate of two percent per month of first cost of car. Toward the close of

the work this was reduced to 1½ percent, and later to one percent of original cost, because as the cars were disposed of it was found that their actual value was more than the book value when depreciated at the higher rate. This more or less arbitrary sliding scale of depreciation no doubt resulted in some unfair charges against the earlier period, but when it is considered that as the cars grew older the expense of repairs (which was charged to operation) was much more, the inequity is more apparent than real.

A volume could be written on cost keeping for operating automobiles and of the difficulties encountered in accurate accounting. In operating a large number of machines it is extremely difficult to get the accurate cost of each, and in reducing this to a unit cost per mile the figures are still less reliable, because mileage is so often estimated or based upon gasoline consumption. The conditions of operation and the results desired vary so widely with different jobs that further discussion will not be attempted here.

Stores and Messes. The five dams are located at considerable distance from towns, so in order to accommodate employees and their families, stores and messes were opened at each dam. Both the stores and messes were managed by a chief steward whose office was at the Taylorsville Dam and to whom the store clerks and cooks reported direct. A butcher shop and bake shop were operated at Taylorsville to serve all five messes and stores. This was known as the "Central Commissary" and all charges to messes and stores went through the central commissary. In the butcher shop was an ice and refrigeration machine. Meats were bought in large quantities and stored at Taylorsville and deliveries made to messes and stores as needed. Other merchandise for sale or use in messes did not actually go through the central commissary, but for record purposes were charged that way. In the beginning, special forms were used to report issues to stores and messes from the central commissary, and the accounting was done from the reports. Later, the standard receiving and issuing blotters referred to on page 58 were put into use to make the accounting uniform. The central commissary plan worked very well, but it involved some duplication in handling supplies, as well as duplication in records, and was discontinued in 1921, when the volume of work had been reduced considerably. Thereafter, meat and bakery goods were bought from merchants in the valley and delivered direct to the dams. Most of the buying for these operations was done through the purchasing department, but some latitude was allowed the chief steward in the purchase of perishable supplies, for which he used the emergency purchase order described on page 54.

The accounting division kept cost for these operations in accordance with the classification shown on pages 29 and 30. Assembling of the cost was done in the same manner as for other operations. At each store was a cash register upon which sales were recorded showing separately cash receipts and coupons redeemed. The field clerk carried the register key and collected the

State of Ohio
MIAMI CONSERVANCY DISTRICT
Department of Engineering and Construction
COMMISSARY REVENUE REPORT

Preture No. ..... Date ..... 19..

Received of ..... Storekeeper,

$ ..... covering the following:

General Store:
Coupons $ .....
Cash $ .....

Mess Hall
Cash Meals .....
Total .....

Cash Register Reading:
Finish—Time ..... $ .....
Start—Time ..... $ .....
Total for Today $ .....

Signed .....
Field Clerk.

I hereby certify that the above items are correct.

.....
Storekeeper.

Storekeeper's Copy

Attach Coupons to This Report and Forward to Accounting Division

..... 19..

..... Storekeeper,

Mess Hall
Cash Meals .....
Total .....

Cash Register Reading:
Finish—Time ..... $ .....
Start—Time ..... $ .....
Total for Today $ .....

Signed .....
Field Clerk.

I hereby certify that the above items are correct.

.....
Storekeeper

Field Clerk's Copy—To be retained in binder.

Attach Coupons to This Report and Forward to Accounting Division

FIGURE 17—COMMISSARY REVENUE REPORT

In addition to the two copies shown above, another carbon was made, which went to the accounting division at headquarters.

cash and coupons and checked the register daily, making a report on the commissary revenue report form (E-140) shown as Figure 17. This report included cash collections at the mess also. Reports were forwarded to the accounting division daily where they were combined with others and vouchers prepared crediting the stores and messes with the amount of the cash and coupons. Each field clerk's cash account (G. L. 1) was debited with the cash, and unredeemed coupons (G. L. 45) with the value of coupons redeemed. Coupons or tickets were not used at the messes. The field clerk or timekeeper appeared at all meals and checked men who were employees, making notation of meals to be charged to each man on the time book and collecting cash from others than employees. These charges were assembled and deductions made on the payroll. The accounting division entries for the credits came from payroll abstracts, referred to on page 48 as Figure 4. In addition to the timekeeper's record of meals, the steward made an independent report of meals served and the two reports were compared as a check on the timekeeper's deductions.

A supplementary account for stores was kept by the chief steward's office. This consisted of a ledger account covering every article delivered. At periods of from three to five months an actual inventory was made at the store to determine the storekeeper's accountability. The following copy of one of these reports of check on store will serve to illustrate the plan:

STATEMENT OF HUFFMAN STORE
FROM APRIL 13, 1921, TO AUGUST 9, 1921

Inventory April 13, 1921	$5,006.01	
Merchandise received	3,554.77	
Total	$8,560.78	
Inventory August 9, 1921		$2,901.83
Cash and coupons to field clerk		3,265.57
Issues to mess		2,134.50
Miscellaneous credits		264.60
Over	5.72	
	$8,566.50	$8,566.50

Some allowances were made for losses in meats, fruit, and other perishable goods, and with these allowances the stores, as a rule, checked a little over. This independent report, with the monthly statement of profit or loss from the accounting division, furnished an approximate control over the store operations. At the close of each month, if an actual inventory of the store stock was not available, an estimate was made by the chief steward's office for use by the accounting division. An entry was made crediting the merchandise account with this estimated inventory and debiting the adjustment inventory account (065E). The opening entry for the next period was a reversal of this. In this manner the total debits against the stores for a given period represented the actual cost of goods sold. Mess inventories were handled in the same manner except that actual physical inventory

was made each month. This was practicable, as the amount on hand was usually small.

Stores were operated at a small profit even though the amount of business was not great. Prices were fixed by the chief steward, the usual mark-up being twenty percent above cost. Effort was made to keep the sale prices near the general market for the same articles in the vicinity.

Messes were operated at a loss. To help in keeping labor during the war period, prices of meals were not advanced as cost of supplies went up. Except for a very short period of about one month, in 1919, the regular price was thirty-five cents a meal for employees and fifty cents for others. In 1921 a flat rate was made to regular boarders of seven dollars a week, and fifty cents per meal was charged to transients. It was impossible to make the messes self-supporting under such conditions, so the loss was charged out to feature cost each month on the basis of direct labor cost.

Camp Maintenance. Camps involving a considerable investment were constructed at all five dams. In addition to the store and mess buildings, bunk houses and dwellings were provided for employees at reasonable rental. The cost of maintaining the buildings, collecting rents, repairing roads, operating the water, sewer, and light systems, and the many miscellaneous expenses incidental to a small town, was assembled in accordance with the classification given on pages 28 and 29. The portion of the investment in camp written off each month was charged to 051X (Depreciation) and formed by far the greater part of the cost. Even by charging a high rate of depreciation, a large part of the cost remained unabsorbed at the finish of the dams. This was partly due to completion of the work ahead of schedule, the estimated rate of depreciation being based upon the first estimate of time required to complete. Two credit accounts were assigned for camp maintenance: one (051W) to take the actual receipts for rentals, and the other (051Y), the portion of the maintenance to be distributed to feature cost. Rents were collected by the camp overseer and turned over to the field clerk who reported the collection to the accounting division and deposited the money, with other revenues, to the credit of the Treasurer. The maximum rental value of each camp was worked out and a report of occupancy was rendered by the camp overseer or field clerk. This report furnished an opportunity for the auditors to check accountability for rent collected.

Excavating Machines. The cost of operating some of the larger dragline excavators was assembled in accordance with the classification shown on page 32. These machines were of large capacity, capable of handling from 50,000 to 100,000 cubic yards of material per month and costing between $5,000 and $10,000 per month to operate. It so happened that electrically operated and steam operated machines were used on similar work, affording an opportunity for valuable comparisons. The assembling of costs furnished an excellent opportunity to study the efficiency of various types of machines and, for like machines on like work,

of gauging the efficiency of crews. The general practice on District work was to charge the cost of these operations direct to the work on which the machines were engaged. This was practicable because each machine was usually engaged for long periods on one feature and item of work. It was also desired to reduce bookkeeping expense to a minimum, so only a few machines were selected upon which separate cost was kept and then only for a comparatively short period. There is some question as to whether it might not have been good practice to operate all the larger machines as production units in the manner described. Comparative monthly reports and charts showing efficiency of machines could have been worked out in such a manner that they would have stimulated interest on the part of crews and superintendents. The extra accounting would not have been expensive, but there would have been the additional expense of making charts and efficiency reports to enable intelligent use of the cost figures.

CHAPTER IX

ACCOUNTING FOR CASH COLLECTIONS AND SALES

Each field clerk was bonded, and advances of small amounts of cash were made to him by the treasurer from time to time, to pay discharged laborers, and small bills for purchases made on the job, such as laundry, milk, eggs, and other supplies bought from

THE MIAMI CONSERVANCY DISTRICT

REPORT OF FIELD CLERK

Date _____

RECEIPTS:		DISBURSEMENTS:	
TREASURER	_____	LABOR	
PAYMASTER	_____	PURCHASES	
STOREKEEPER	_____	EXPENSES	
MISCELLANEOUS (LISTED)		TREASURER	
		MISCELLANEOUS (LISTED)	
	_____		
			$
Balance 19	$	Balance _____ 19	$
TOTAL	$	TOTAL	$

I certify the above statement to be correct in all respects.

_____ Field Clerk

APPROVED:

_____

REMARKS:

FIGURE 18—MONTHLY SUMMARY MADE BY FIELD CLERK

farmers and peddlers and delivered to the camp. This advance was deposted by the clerk in some bank near the site of the work to the credit of the "Miami Conservancy District by ..... Field Clerk." The field clerk's payments were made by check on this account. His collections were deposited in the same account but kept separate from other receipts by a special column in his cash book. He was not allowed to draw checks on the collections, these being remitted to the treasurer. The payments for labor were entered as advances on the payroll and the field clerk was reimbursed by the paymaster at the regular pay period. A state-

REMITTANCE LETTER

THE MIAMI CONSERVANCY DISTRICT

_____ DEPARTMENT

TO THE TREASURER:—

Date _____

We attach hereto $ _____ as follows:

Cash - - - - - - - - - $

Money Orders - - - - - $

Checks - - - - - - - - $ _____

TOTAL - - - - - $

RECEIVED FROM:

CREDITS:

I HEREBY CERTIFY that the transactions recorded above are correct in all respects, and that the amounts involved represent the reasonable maximum obtainable by the Miami Conservancy District.

_____

_____ Title

To the best of my knowledge and belief the attached remittance is correct.

_____ Department Head

Approved:

_____

_____

FIGURE 19—REMITTANCE LETTER

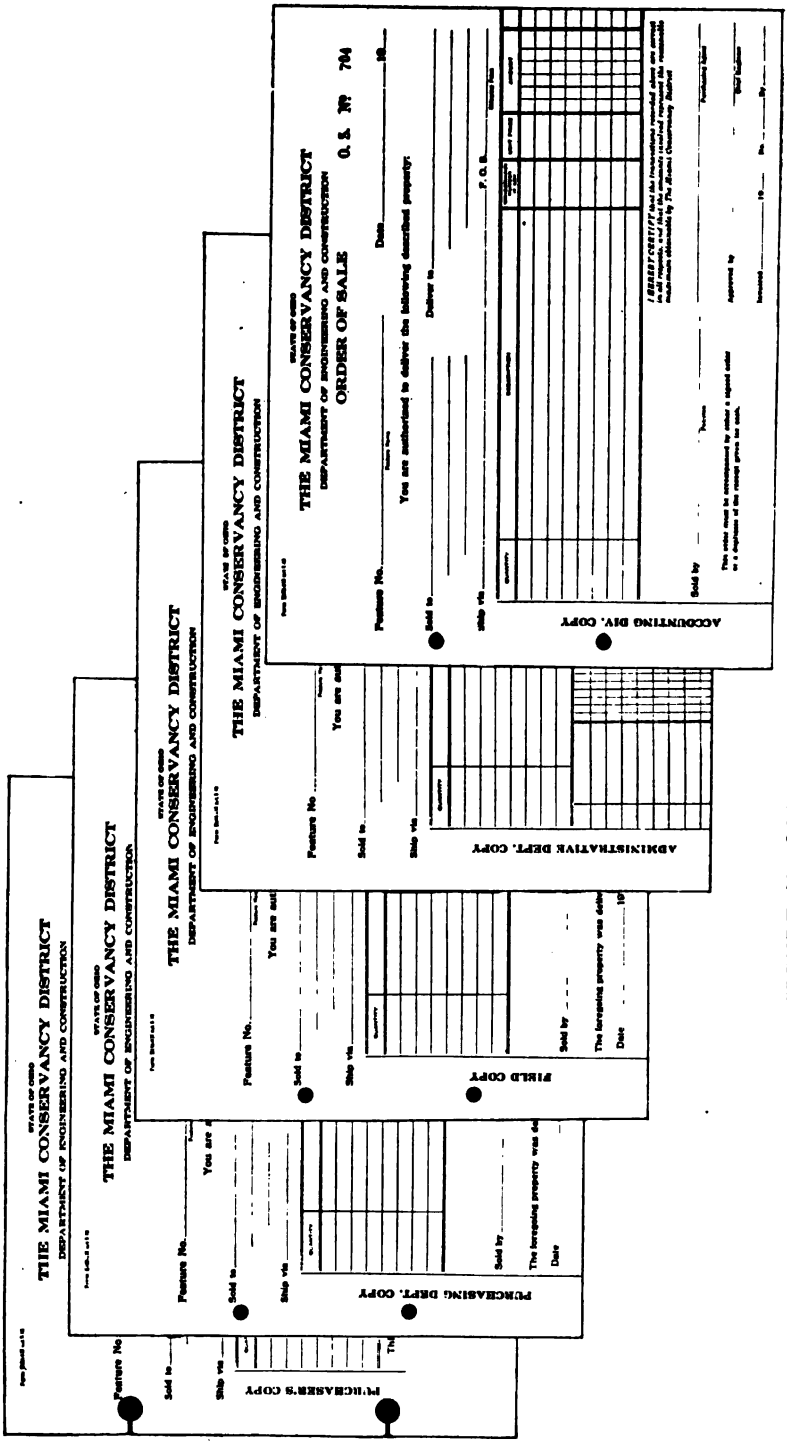


FIGURE 20—ORDER OF SALE, FORM E-185
The various copies were differentiated by color.

ment of other expenses, accompanied by receipted bills as vouchers, was rendered monthly. This was checked in the accounting division, charged to appropriate accounts and forwarded to the treasurer for reimbursement to the field clerk. The field clerk was responsible for all cash collected at the camps. This included sales at stores and cash meals at messes (discussed on page 75), other sales to employees where no store was operated, and collection of rents. Collections were reported daily on the revenue report form (Figure 17), from which the accounting division obtained the credits for store and mess. The standard form of cash book was used by the field clerk. His accounts were audited by a representative of the treasurer's office periodically and he was responsible for his imprest fund to the treasurer's office. He was charged on the treasurer's books for funds advanced and was reimbursed from time to time, being credited with remittances of collections and with amounts paid as shown on monthly expense statements. A summary report on form T-5 (Figure 18) was made monthly by the field clerk showing receipts, disbursements, and balance.

General Sales. As the work drew to a close, much equipment and surplus material was made available for sale. A sales division was organized under the general direction of the purchasing agent who worked in close coöperation with the salvage division. A large part of the time of the chief of the salvage division was devoted to sales. Other sales agents were appointed and selling activities greatly increased during the last year of construction. This involved the handling of money by a number of persons, as most of the smaller sales were made for cash.

Large sales were made upon special authority by use of a form called "Order of Sales" (Form E-185, Figure 20). When equipment or materials were sold, they were listed on this form with the terms of sale and delivery, and the document was used as a contract of sale. The original went to the accounting division after approval by the chief engineer, and upon delivery of the material invoice was issued against the buyer. Invoices were numbered serially and entered in the sales register, debiting accounts receivable (G. L. 21) and crediting the proper inventory or feature account. Sales orders were not used for the smaller transactions or miscellaneous cash sales. Neither were invoices issued covering small cash sales. These were taken into accounts receivable, however, under the title "Cash Sales" and the names of all persons to whom sales were made were listed in the "Accounts Receivable" ledger. The remittance letter transmitting such cash to the treasurer, when put through the journal, gave credit to accounts receivable. Authority for sale was contained in lists published from time to time as surplus supplies and material became available.

A special form was printed as a bill or sale slip, and every one making a sale, whether for cash or charge account, filled out one of these slips. A copy was handed the purchaser, receipt being given if cash was received, a copy was kept by the sales clerk, and a copy accompanied the remittance letter to the treas-

urer. Cash was remitted daily. The form of remittance letter used is shown by Figure 19. Upon it was listed each sale and all information necessary for accounting purposes. When the cash was received in the treasurer's office, a copy was returned to the accounting division for proper entry. A copy of the sale slip was attached to the copy of the remittance letter in the accounting division's file as the permanent accounting record Entry was made to the "Journal and Cash Receipts Register" from this copy of the remittance letter.

CHAPTER X

BOOKS OF ACCOUNT

General Ledger. The accounts kept in the general ledger are described in Chapter IV, pages 35 to 42. A Kalamazoo binder No. B. S. B. 472, sheets 11 x 11, was used. Figure 21 shows the ledger ruling used. No details were carried to the general ledger, but some of the accounts were subdivided when occasion required. Most of the accounts carried in this ledger were controlling accounts and details were shown in subsidiary books. There was nothing unusual in the handling of the general ledger

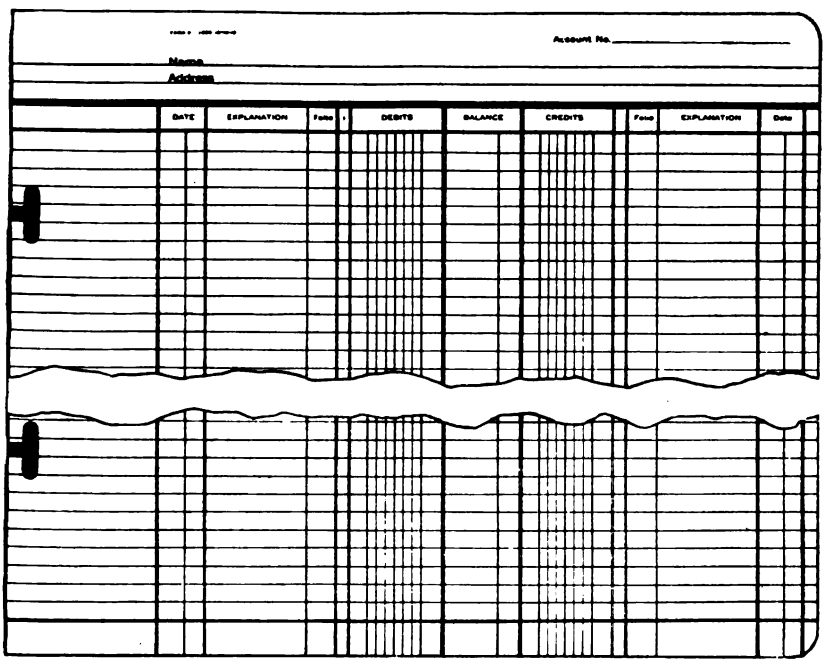


FIGURE 21—GENERAL LEDGER SHEET

except that the general classification sheet (Figure 6) was used to assemble the accounts for the ledger, and postings were made in total from these classification sheets as described on page 50. This made the ledger less bulky and more easily kept, but, on the other hand, checking back to original sources of entry involved going to the general classification sheets for references. Some criticism was made of this system by one of the auditors, but it was found to be very practicable and a labor-saver.

Cost Ledgers. Three cost ledgers were used, one each to support the controlling accounts—G. L. 31, "Feature Cost;" G. L. 32,

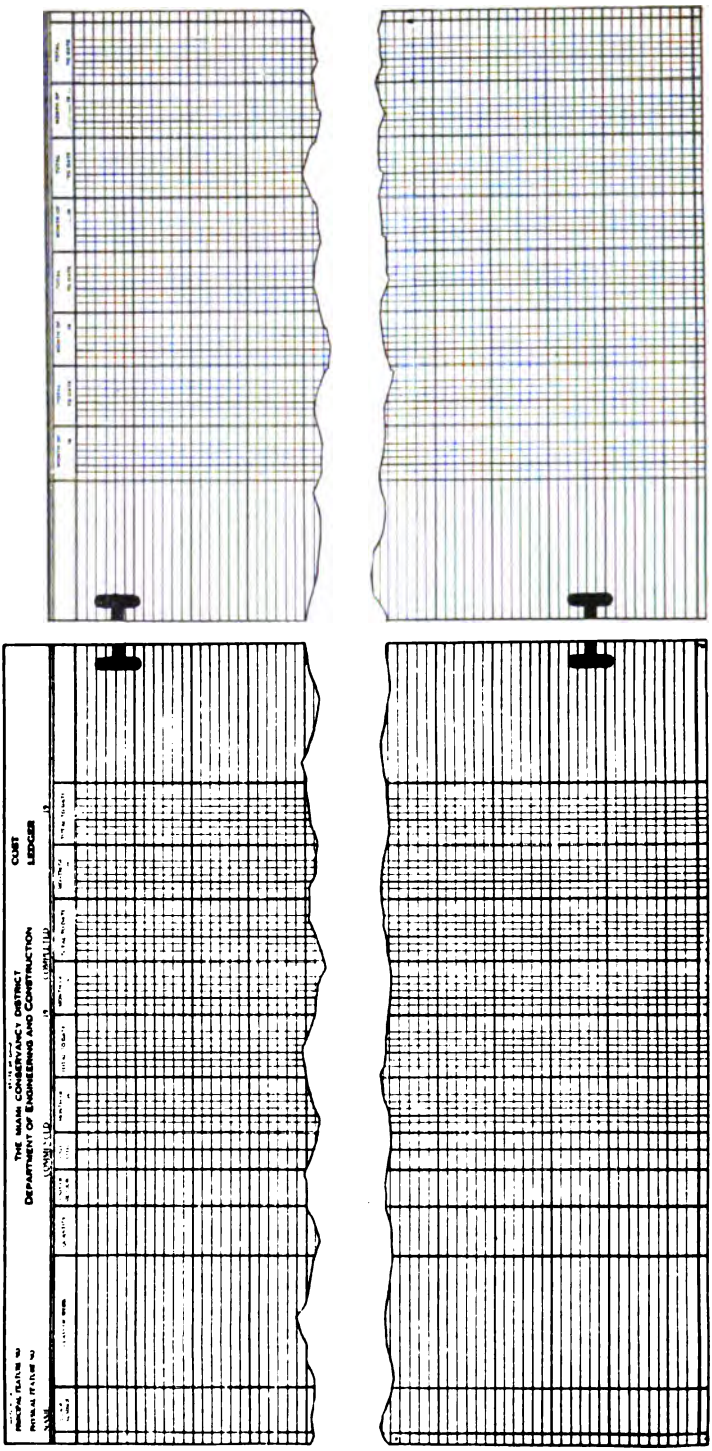


FIGURE 22—COST LEDGER SHEETS

The sheet on the right was ruled on both sides, and any number of sheets could be added, thus providing for an indefinite continuation of the cost record.

"Plant Cost," and G. L. 33, "Operation or Clearing Accounts." Kalamazoo binders (No. B 556797), taking sheets 11 x 14 were used. The Cost Ledger sheets are shown by Figure 22, Form E-4. This is not, technically speaking, a ledger sheet, in that there is not a separate debit and credit column. It took balances to accounts only and was posted from the recapitulation sheet shown as Figure 5, described on page 50. For details and reference back to original source of entry, it was necessary to go through the recapitulation sheets. These sheets were made into a book and filed each month to support all business for the month. It will be noted that a sheet in the cost ledger was assigned for each feature or job corresponding to a primary account number, and a line on the sheet was used for each item of work corresponding to a secondary account number or letter. Debit balances in any account or on any line were shown in black ink and credit balances in red ink. In case of the operating accounts showing both debits and credits, usually the debits were entered first and footed, then the credits, and the balance carried to the foot of the column. Two columns were used for a month's business, one for the cost for the month and one to carry forward totals to date. Insert sheets were provided without any column for names of accounts and without column headings, so that the same account could be carried forward indefinitely without transferring to another page. This is a big advantage for construction work such as the District did, when accumulating totals are necessary for ready reference. Upon completion of a job, the quantity of work performed and unit cost may be entered. A column might easily be assigned for estimated cost and these sheets would then be a complete history of the cost no matter how many months or years it took to do the work.

Accounts Receivable Ledger. Details were kept for all accounts receivable, in separate ledgers, for controlling accounts in Group III, G. L. numbers 21 to 25 inclusive. The same sheet (Figure 21) was used as for the General Ledger. No special description is necessary for these details except to note that all "Cash Sales" which were run through G. L. 21 were entered in one account by listing names of buyers on both debit and credit sides. A schedule of the balances in each account was submitted at the close of the month to support the balances in the various general accounts.

The Invoice Register. An 11 x 14, specially ruled sheet was used for the registering of invoices. (See Figure 23, Form No. E-2.) Current sheets were carried in temporary binders and these were bound permanently at the end of each year. The invoice register used was a columnar book of original entry, designed to minimize as much as possible the accounting for purchases and miscellaneous invoices received. It had columns for number, date, name, and the usual posting references. It was self-balancing, debit columns being assigned for accounts payable, inventories, and direct charges. Credit columns were assigned for the two adjustment accounts, G. L. 19, "Goods in Transit," and G. L. 15, "Inventory Adjustment," and for "Miscellaneous." Im-

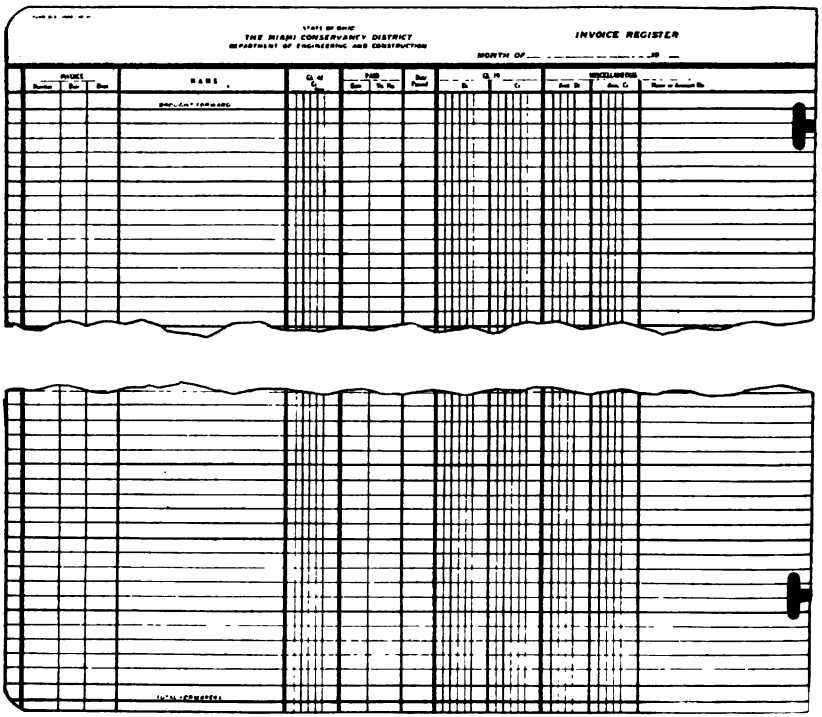


FIGURE 23—INVOICE REGISTER, FORM E-2
This was a double page form, set in the binder as shown above and on opposite page.

mediately upon receipt of an invoice it was referred to the accounting division where it was numbered and entered in the register. The date of entry and date of invoice were put in the date column. The total of the invoice was a credit to accounts payable and was entered in the column marked "G. L. 42." If it had already been approved for payment and was ready to voucher, a direct charge was made to the account specified, by entry in one or more of the columns for "Direct Charges." Most of the invoices received had not been checked, so they were charged to "Goods in Transit" in the column marked "G. L. 19" and then referred to the purchasing agent, if a purchase, who checked them with his orders (see page 98) and if he found them correct passed them for payment. Voucher request was then made on the treasurer and when payment was made the date and voucher number were entered opposite the first entry. As explained on page 60, the credit to G. L. 19 came from the warehouse receiving blotter on which the charge was taken into inventory. Sometimes an invoice was cancelled, in which case it was entered in red in the invoice register, debiting accounts payable in credit column "G. L. 42" and placing the credit in the credit column for G. L. 19.

The following example illustrates the movement of an invoice by the system used: An order is placed on which the price is

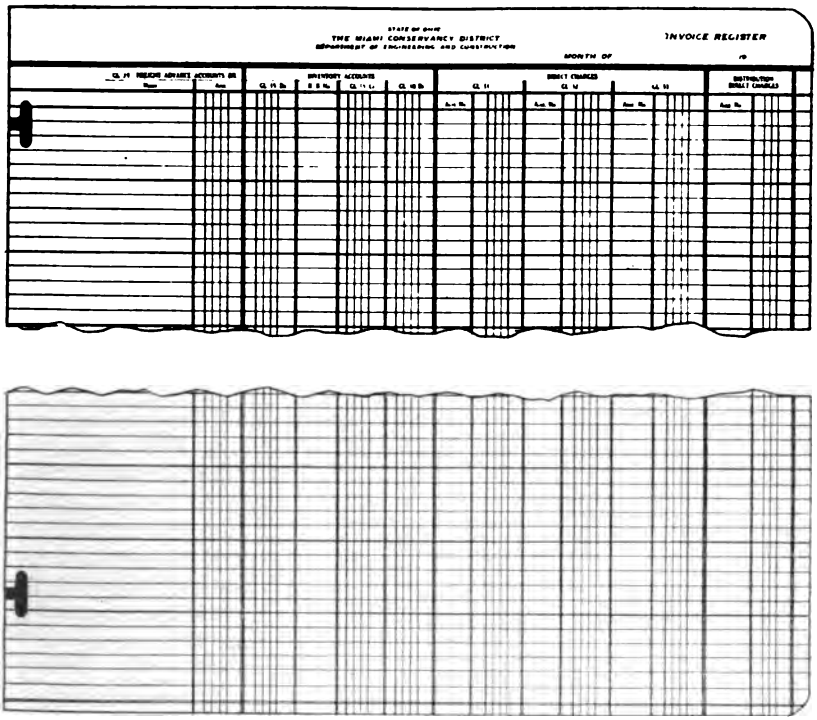


FIGURE 23—INVOICE REGISTER, FORM E-2

This was a double page form, set in the binder as shown above and on opposite page.

estimated at $250 to be charged to stock in warehouse (G. L. 16). The invoice when presented is for $275 and goes to the invoice clerk who enters it in the invoice register.

Dr. G. L. 19 "Goods in Transit".....$275.00
Cr. G. L. 42 "Accounts payable" $275.00

In the meantime, the article is received at the warehouse and taken up on the receiving blotter in the regular manner at the estimated price shown on the purchase order ($250), the blotter number being noted on the inspector's copy of the order and entry being made in column G. L. 16.

Dr. G. L. 16 "Warehouse Stock".....$250.00
Cr. G. L. 19 "Goods in Transit" $250.00

When the invoice is passed for payment, it again comes to the accounting division. Comparison is made with the entry on the receiving blotter, the invoice number being noted in the "check" column and the blotter number being noted in the invoice register with the date passed. The entry is—

Dr. G. L. 15 "Inventory Adjustment" $25.00
Cr. G. L. 19 "Goods in Transit".... $25.00

This clears "Goods in Transit" and the use of adjustment account, No. G. L. 15 (see page 35), will make it unnecessary to change the price as originally entered on the receiving blotter.

If there is no difference between the amount of the invoice and the amount taken up on the receiving blotter, the only entry in the invoice register when the invoice is passed for payment is a memorandum in the reference columns "Date passed" and "R. B. No.," the latter referring to receiving blotters from which information was obtained.

When the voucher is paid, entry is made in the disbursement voucher register.

Dr. G. L. 42 "Accounts payable".....$275.00
Cr. G. L. 61 "Disbursement Vouchers" (Cash) $275.00

At the same time that entry is made in the disbursement register, the date paid and the voucher number are entered in the invoice register on the same line as the original entry of the invoice.

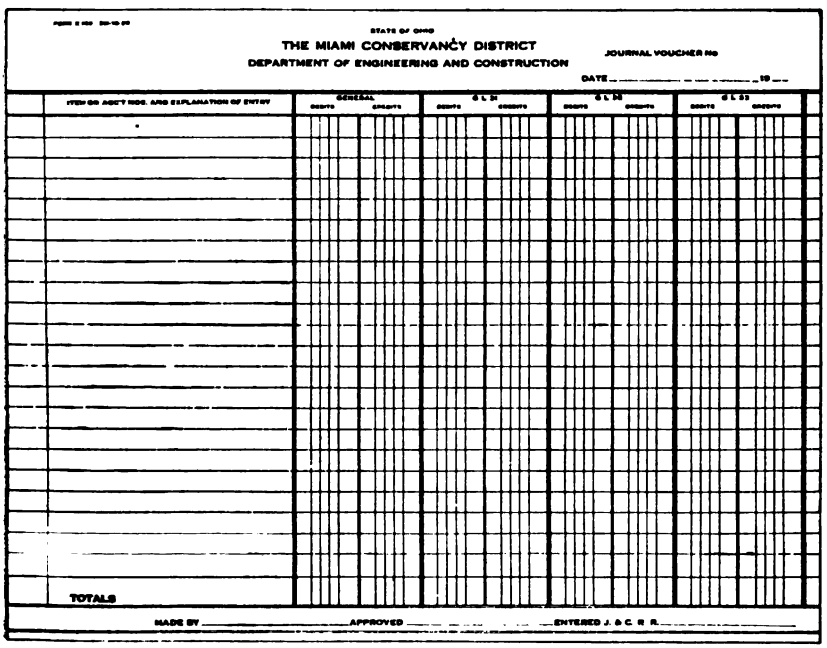
Sometimes, invoices were passed which had been taken into the invoice register in previous months. Then, if adjustments were to be made, the second entry could not be placed on the same line as the first entry because the register was closed every month. The subsequent entry was made with the current month's business, indicating the same invoice number which served as reference to the first entry. These subsequent entries were made in columns for G. L. 19 and G. L. 15 only. The fact that no entry was made in column G. L. 42 and that the vendor's name was left out, indicated that it was a "hold-over" adjustment. These adjustments were shown on separate sheets, placed at the end of the record for the month's business.

The column on the extreme right of the invoice register headed "Distribution Direct Charges," was used merely as a recap of these charges, for convenience of the bookkeeper in posting.

No accounts payable ledger was kept, the open items on the invoice register equalling the balance in controlling account G. L. 42. This saved a great deal of detail posting. On the other hand, it would have been a convenience, sometimes, to know how the account stood with certain firms with whom a large volume of business was done. It certainly would have been a waste of effort to run all purchases through a special ledger, for many of them were from persons with whom no regular business was done. Probably a middle ground would be a good stand to take on this subject, carrying only the larger accounts to a special ledger and creating a special control for them.

Journal and Cash Receipts Register. This journal was to record the initial entry of all cash receipts and all other entries for which no special register was provided. The sheet was 11 x 14, as illustrated by Figure 25 (Form No. E-11). The documents from which entries were made to this record were designated journal vouchers and were numbered consecutively by months. The journal voucher number indicated the year of construction, the month of the year, and the number of the voucher in the month. Thus, No. 4-B-25 would be the twenty-fifth voucher in the month of February for the fourth year after beginning operation, or in this case 1921, as the first year of construction was

1918. The standard journal voucher form is shown by Figure 24 (Form No. E-169), but payroll abstracts, remittance letters, engineers' estimates of contract earnings, and other papers, were used and numbered as journal vouchers. These vouchers were filed in vertical folders by months to support the entries made in the journal. The various bookkeeping operations previously referred to in this report were assembled in shape to enter as journal vouchers. The bookkeeping work of the office was so arranged that different clerks could work on different operations, so that everything would be ready for entry in the journal shortly after the close of the month. Thus, one clerk handled labor, another warehouse issued blotters, another warehouse receiving


STATE OF OHIO
THE MIAMI CONSERVANCY DISTRICT
DEPARTMENT OF ENGINEERING AND CONSTRUCTION
JOURNAL VOUCHER No
DATE _____ 19__

	ITEM OR ACCT NOS. AND EXPLANATION OF ENTRY	GENERAL		G. L. 21		G. L. 22		G. L. 23	
		DEBITS	CREDITS	DEBITS	CREDITS	DEBITS	CREDITS	DEBITS	CREDITS
	TOTALS								

MADE BY _____ APPROVED _____ ENTERED J. & C. R. R. _____

FIGURE 24—STANDARD JOURNAL VOUCHER, FORM E-169

blotters, another the invoice register, etc. Sometimes, depending on the volume of business, one clerk handled more than one operation, but the work was so organized that it could be readily split up and parts assigned to as many clerks as necessary to get the work done promptly. To facilitate reference to vouchers, uniform numbers were given to the same operation for different months. The following is a list which will serve to illustrate the number of regularly recurring operations:

Voucher No.

1 General Warehouse clearance.
2 Warehouse issues at Franklin and Troy.
3 Englewood depreciation list.

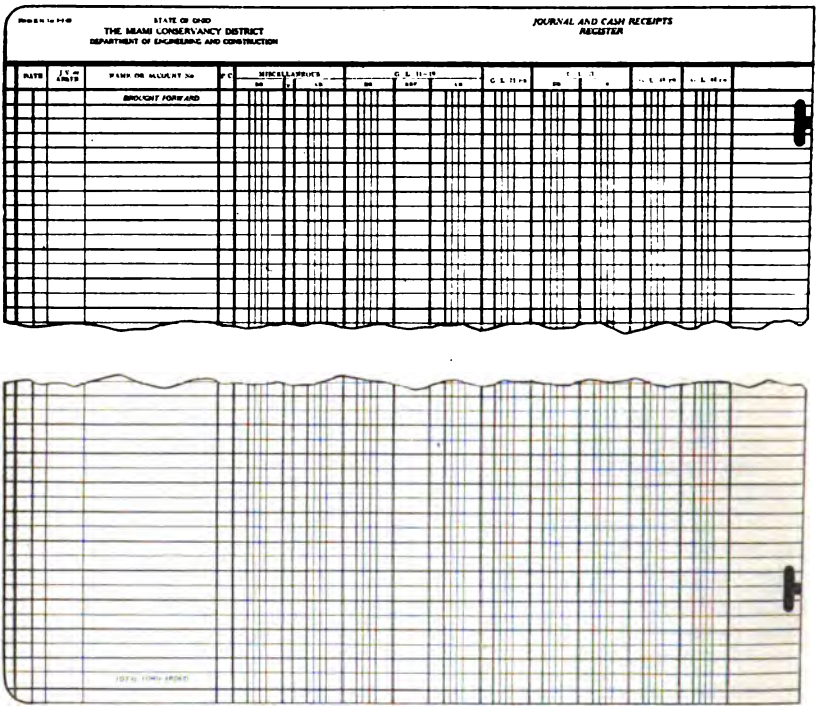


FIGURE 25—JOURNAL AND CASH RECEIPTS REGISTER, FORM E-11

This was a double page form, set in the binder as shown above and on opposite page.

4 Taylorsville and meat shop depreciation list.
5 Huffman depreciation list.
6 Tippecanoe City depreciation list.
7 Piqua depreciation list.
8 Troy depreciation list.
9 Dayton river channel depreciation list.
10 Franklin depreciation list.
11 Hamilton depreciation list.
12 Miscellaneous depreciation list.
13 Shop (Machine)
14 Garage (Motor vehicles)
15 General Headquarters expense distribution.
16 General superintendence expense distribution.
17 Sand and gravel from Dayton screening plant.
18 Revenue reports.
19 Mess and store inventories.
20 Clearance of Warehouse operation expense.
21 Charges from Garage.
22 Recapitulation of Machine Shop orders.
23 Transfers between features.
24 Power and light distribution.

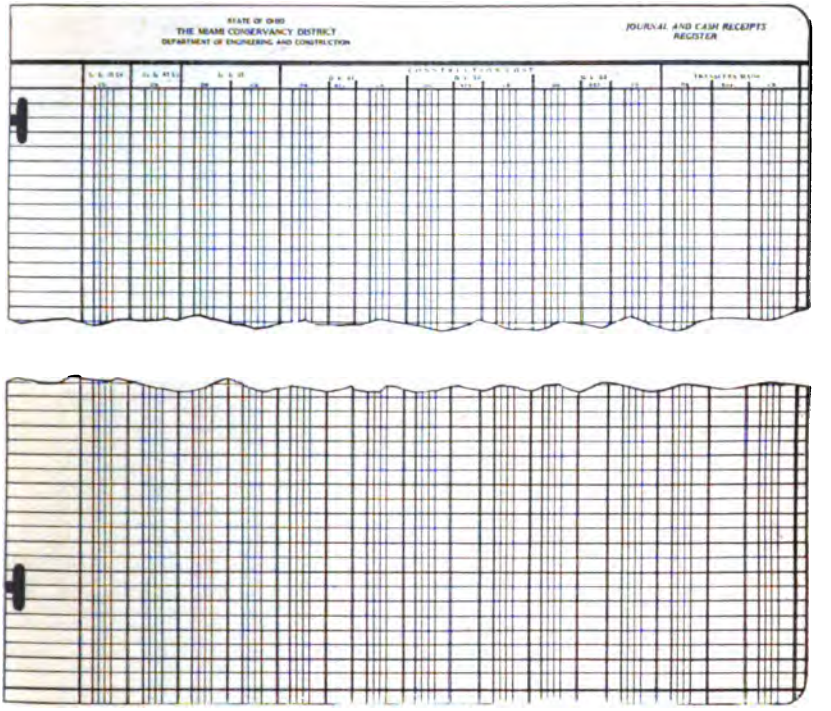


FIGURE 25—JOURNAL AND CASH RECEIPTS REGISTER, FORM E-11

This was a double page form, set in the binder as shown above and on opposite page.

25 Salvage operations clearance.
26 Dragline machine operation clearance.
27 Transfer of maintenance costs.
28 Administrative Dept. credit for maintenance.
29 Administrative adjustments.
30 Opening entries for store and mess inventories.

In addition to these there were, of course, many more special vouchers put through the journal. In the journal, entries were made to controlling accounts only. The bookkeeper who kept the journal entered the vouchers in total as they affected the controlling accounts, carrying debits and credits to the accounts as indicated by column headings. He then passed the vouchers to the clerk who assembled cost on the recapitulation sheets (Figure 5) for entry in the subsidiary ledgers. Before posting the details to subsidiary ledgers, a trial balance was taken to see that the details agreed with the controlling accounts. The total debits and credits to the various controlling accounts were obtained, as before stated, by the use of the general classification sheets (Figure 6). Entries to these general sheets were made from other registers also, column totals only being posted. The cost clerk posted details from other registers, also, when any entries were

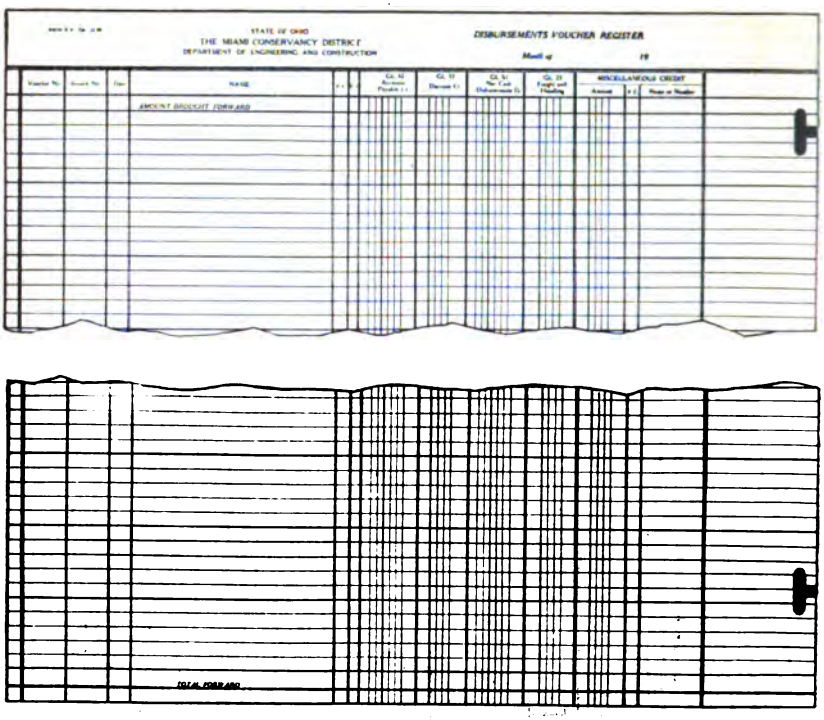


FIGURE 26—DISBURSEMENTS VOUCHER REGISTER, FORM E-3

This was a double page form, set in the binder as shown above and on opposite page.

made therein as direct charges, but by far the most of the cost clerk's figures came from the details on the vouchers.

The journal voucher forms were ruled with four double columns, one for miscellaneous items and one for each cost ledger (31, 32, and 33), to facilitate posting to controlling accounts. If entries were carried to wrong columns on the journal voucher, and the general bookkeeper did not catch the error, the detail and control would be out of balance.

It probably would have been better to keep the general journal separate from the cash receipts register, and the scheme was originally so designed. Combination was made when cash receipts were small and the matter was hardly of enough importance to warrant a change later. The cash receipts portion of this register corresponds to the debit side of the cash book in commercial accounting.

Disbursement Voucher Register. Figure 26 illustrates the disbursement voucher register which was used to record cash disbursements only. It is an 11 x 14 sheet for the same binder as the other registers. Columns provided were for the general ledger accounts most frequently affected by disbursements, with

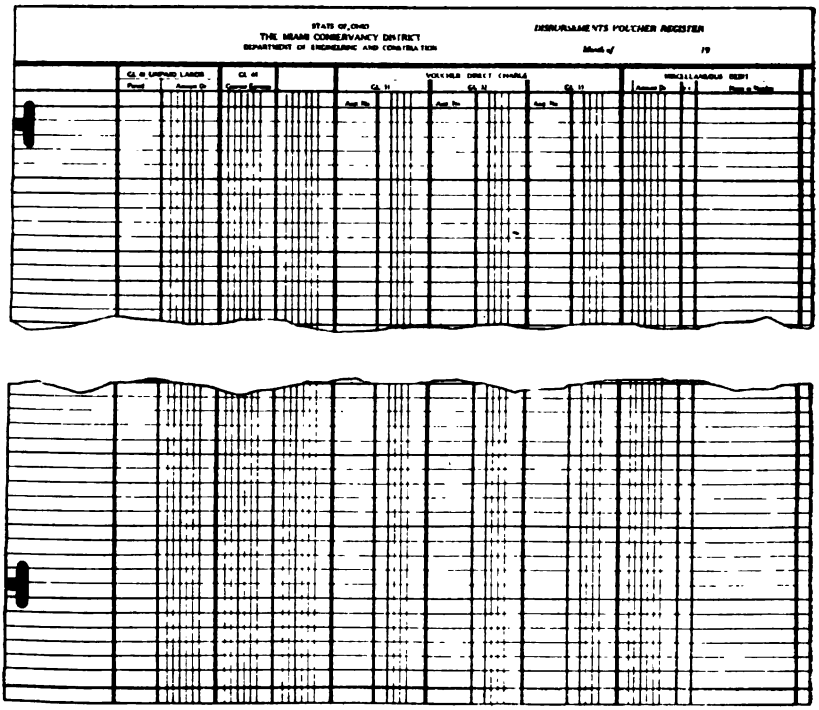


FIGURE 26—DISBURSEMENTS VOUCHER REGISTER, FORM E-3

This was a double page form, set in the binder as shown above and on opposite page.

special columns for "Miscellaneous Debits and Credits" and three columns for direct charges to cost ledger accounts (G. L. 31, 32, and 33). The total of each payment was credited to G. L. 61, "Disbursements," and was the amount of the District's investment in cash for construction of the flood works. This register corresponded to the credit side of the cash book in commercial accounting, and the cash receipts portion of the journal was the debit side of the cash book. The separation was made primarily because by the use of a separate register more space was provided for columns and secondarily to enable the cash book to be handled by different clerks.

It was through the disbursement and cash receipts registers (the cash book) that the accounts of the Department of Engineering and Construction were reconciled with the books of the treasurer. At the close of every month's business, the total amount of paid vouchers entered as a credit to G. L. 61 in the disbursement register was compared with the total charged against the department by the treasurer, and, if any differences existed, the errors were immediately located and the accounts brought into agreement.

CHAPTER XI

PAYMENT OF BILLS

The voucher system was used in the District's accounting throughout. All payments were made by the treasurer's office except in cases where advances were made to bonded employees for petty cash and emergency payments. Such employees were virtually under the treasurer's office, so far as their petty cash account was concerned.

After bills incurred by the different departments were approved for payment, a "voucher request" (Form No. E-28) was prepared, usually in the purchasing or accounting divisions, although any one authorized to approve claims against the District

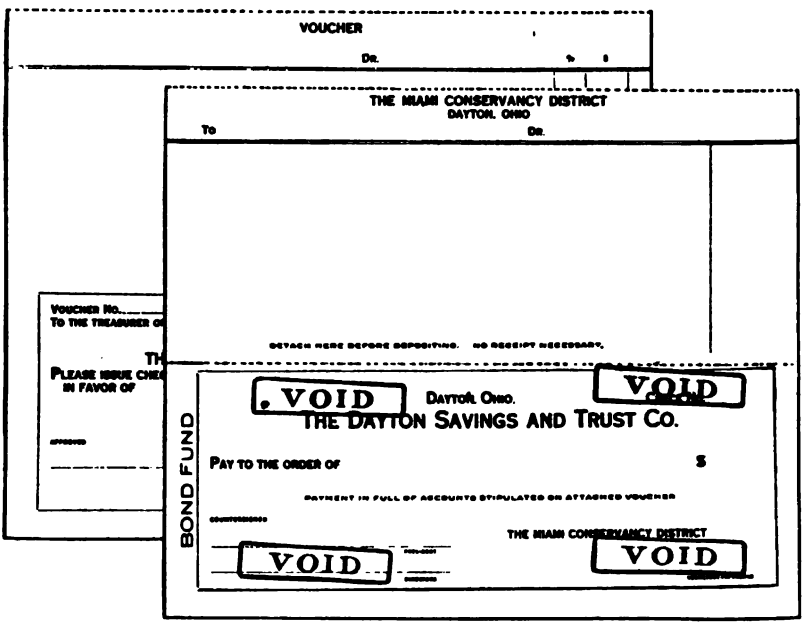


FIGURE 27—VOUCHER CHECK

might and did prepare requests. Figure 28 is the form used in the construction department. It was an order on the treasurer to pay certain bills listed thereon. For accounting purposes, several copies of these requests were made. The original, upon which certification and approvals appear, was kept in the treasurer's file. One copy, without signatures, was mailed to the payee, with the check, so that he could identify the payment with his invoice. One copy (yellow), to which were attached all the invoices and supporting papers, was returned by the treasurer to the accounting division and was the basis of entry in the dis-

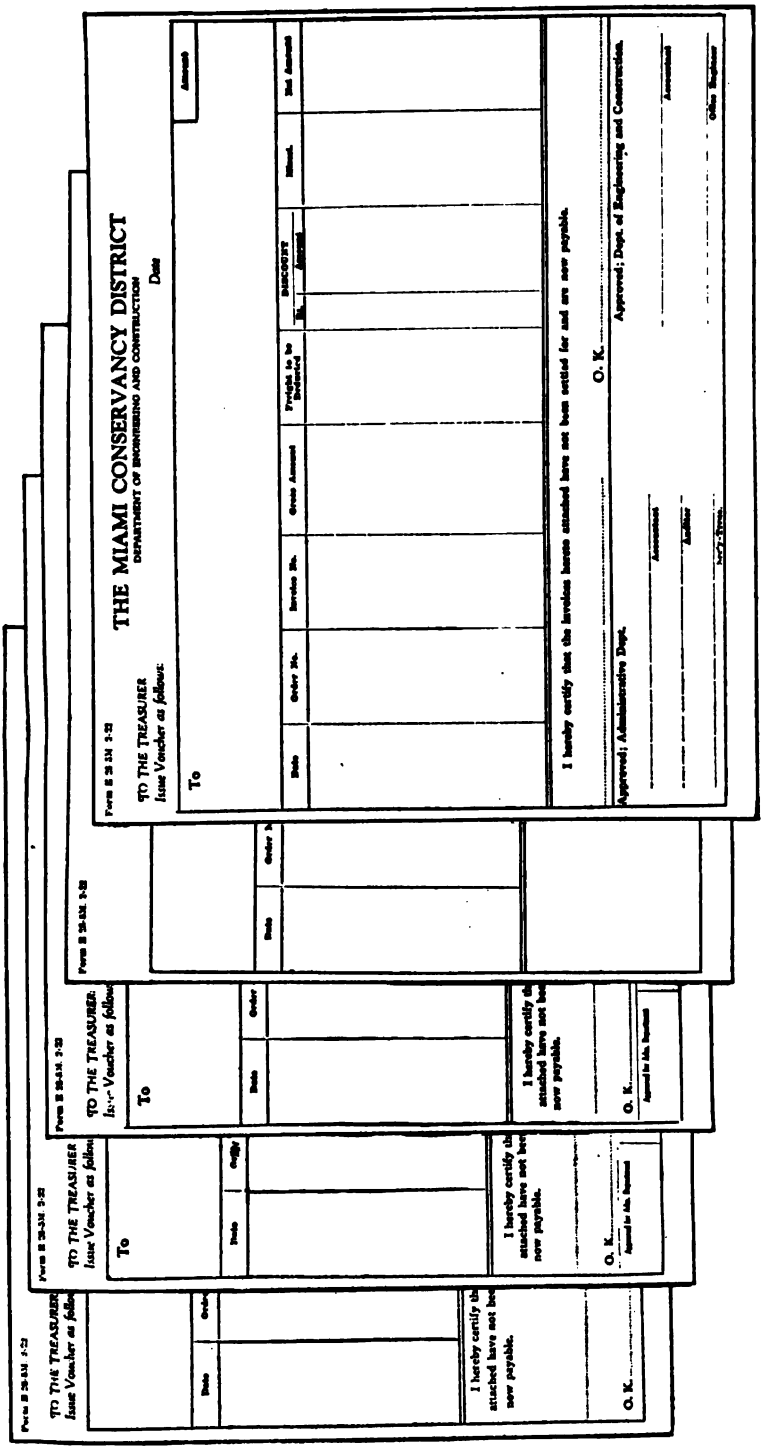


FIGURE 28—VOUCHER REQUEST USED BY ENGINEERING DEPARTMENT, FORM E-28
The various copies were differentiated by color.

bursement voucher register. One copy (pink) was detached by the accounting division when the request went through, and served as a tickler until the yellow copy came back with the date of payment and voucher number indicated thereon. The fifth copy was not always necessary, but was made up and usually kept by the purchasing agent or other person who prepared the request.

The treasurer's office prepared the check and mailed it to the payee. Figure 27 is a copy of the voucher check. All checks except those issued by the paymaster for labor, and by field clerks from the imprest fund, were signed by the treasurer and approved by the board of directors. This method involved some duplication and delay in payment. The voucher request and voucher check might have been combined with some advantage, but the requests originated from many sources more or less distant from the treasurer's office, and the treasurer's funds were in separate banks and accounts, about which the departments had no information, so it was not considered practicable to combine the forms.

The checking of the invoices was done by the bill clerk in the purchasing division, at the time of passing for payment. Another check was made in the accounting division. Computations and extensions were not checked by the treasurer's office except to see that the totals on the voucher request equalled the total of invoices attached.

When the invoice was first registered by the accounting division, it was stamped as follows:

Inv. No. .....
Acct. No. .....
Quantity O. K. .....
Price O. K. .....
Quality O. K. .....
Terms O. K. .....
F. O. B. .....
Amount .....
Extensions
and Footings O. K. .....
Delivered to .....
.....
.....

The number which was then assigned served as a reference in the future handling of the invoice. The request on the treasurer, listing invoices for payment, carried the certificate of the purchasing agent or other person having personal knowledge of the transaction. Two approvals were added for the department, one by the accounting division and one by the office engineer. In the treasurer's office it was approved by the bookkeeper, auditor and treasurer. A time stamp system was used to record the times at which requests passed the various divisions.

CHAPTER XII

FILES

The question of filing is important. In the average office much time is lost in locating documents after they have been filed. In accounting this is particularly important, because the final accounts show results in totals, and in order to verify or obtain details, resort must be had to supporting papers filed elsewhere. All the papers and correspondence relating to accounts of the department were filed in the accounting division. Sectional vertical cases were used.

Correspondence was filed alphabetically and as nearly as possible by subject. Thus the correspondence with a division engineer would be under his name, and all that about a particular matter was together. This was not always satisfactory, because the tendency in such a system is to place the letters in date order, and when any particular subject is to be investigated the whole folder has to be gone over. The correspondence was not extensive, so this defect was not serious.

Receiving and issuing blotters, covering warehouse operations, were filed numerically, by months, each job's blotters being kept separate. They were bound in folders fitting the sheets and labeled in large letters.

Original bills showing issues from the main warehouse were filed numerically. Transfers were filed in the same manner.

Equipment cards were arranged by plant account and feature, with a cross index on small cards for the whole, arranged alphabetically by name of equipment.

Journal vouchers were filed by months numerically with all supporting papers attached, except for depreciation lists which were filed separately with the equipment records.

Disbursement vouchers were filed alphabetically by years, all payments to one firm being in a folder. Invoices were attached to the vouchers.

Card records were kept for contracts and these were filed by agreement numbers. A cross index by use of a small card with less information thereon, was filed alphabetically under the contractor's name.

Cost reports were filed by months, all of the regular reports and schedules being bound with the balance sheet.

Special reports were filed in a miscellaneous file.

Bound books were kept in a safe cabinet when not in use.

The aim in the accounting division was to file all records so that accounts could be traced to the original source of information with the least possible effort and delay. There is no doubt but that all papers supporting the books of account should be kept as close as possible to the books in the accounting office.

About the correspondence there is some question. It would seem that all outside correspondence should be filed in a central

filing section for all divisions. Much depends upon the organization and the physical location of the various divisions. In the District organization, every division kept its own files, with the result that there was duplication, and sometimes letters that belonged in the general file got into some division file and vice versa. Probably more filing cases were used than a central file would have required. Extra copies of letters were made so that each file concerned could have a complete record. In small divisions, individuals whose time was valuable worked on files, whereas all the work might have been done by a specialized central file clerk. Letters on the same subject would be found in different divisions and it was difficult to know when the matter relating to a single case had been assembled. These objections were offset somewhat by the undoubted advantage of every responsible head of a division having his records close at hand.

CHAPTER XIII

STATEMENTS AND REPORTS

Financial statements were made monthly. They consisted of a balance sheet, covering all general accounts, supported by schedules showing details. At first the balance sheet was written on plain paper, but the latest reports were a printed form, E-214, illustrated by Figure 31. The schedules for accounts receivable and accounts payable were merely itemized lists of amounts due from and to individuals and firms. Each month, adding machine tapes of balances shown on warehouse cards were made. Warehouse reports consisted of these tapes and a report showing in totals:

1. Balance at the beginning of the month.
2. Amount received during the month (total of Receiving Blotters).
3. Amount issued (total of Issue Blotters).
4. Balance.

This was compared with the tape total and general ledger balance, and served as the means of checking the warehouse accounts.

Cost reports were rendered on various forms from time to time, at first on plain paper, but finally on standard forms, a different color for each class of cost—pink for feature cost, white for operating accounts, and green for plant costs. Figure 29 illustrates the feature cost report. Standard item accounts were printed, so as to save typing. Not all the spaces were required for all features, but this did not injure the value of the reports in any way. A sepcial form was made for railroad costs, and blanks without the printed names of accounts were used for miscellaneous work. Figure 30 illustrates the forms used for plant and operation reports. Variation in accounts for the two latter reports was so great that only headings were printed on those forms.

The cost reports were copies of the cost ledgers, with additional columns for quantity of work done, and unit cost for the month and to date. Theoretically they should have been prepared from the ledger after it had been posted and balanced, but in practice, reports were written up direct from the recapitulation sheets (Figure 5) and the postings were made to the ledger later. Reports for a month's operation were available for issue by the eighth to the tenth of the following month. Copies were sent to each division engineer's office together with a summary of unit costs and quantities, which gave comparisons between features. The latter was prepared on tracing cloth by the office engineer and blue prints were mailed to field offices.

The budget report consisted of a comparison of the total feature cost at any time with the amounts authorized for that feature and the balance available for completion. A glance at this report showed the financial status of each job on a cost basis.

Special and comparative statements were made from time to time, but the foregoing were the most important regular reports.

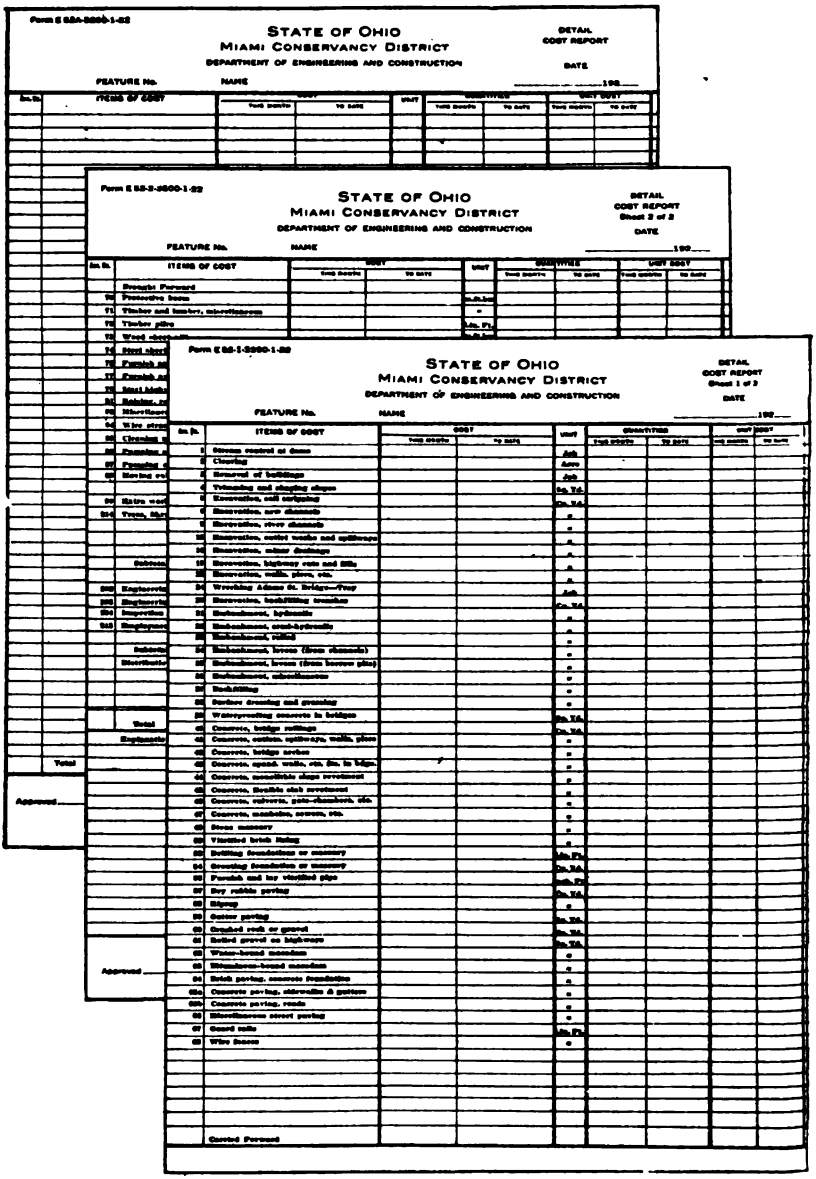


FIGURE 29—COST REPORT SHEETS

The first two were for the features with standard items of work, the third was for miscellaneous work.

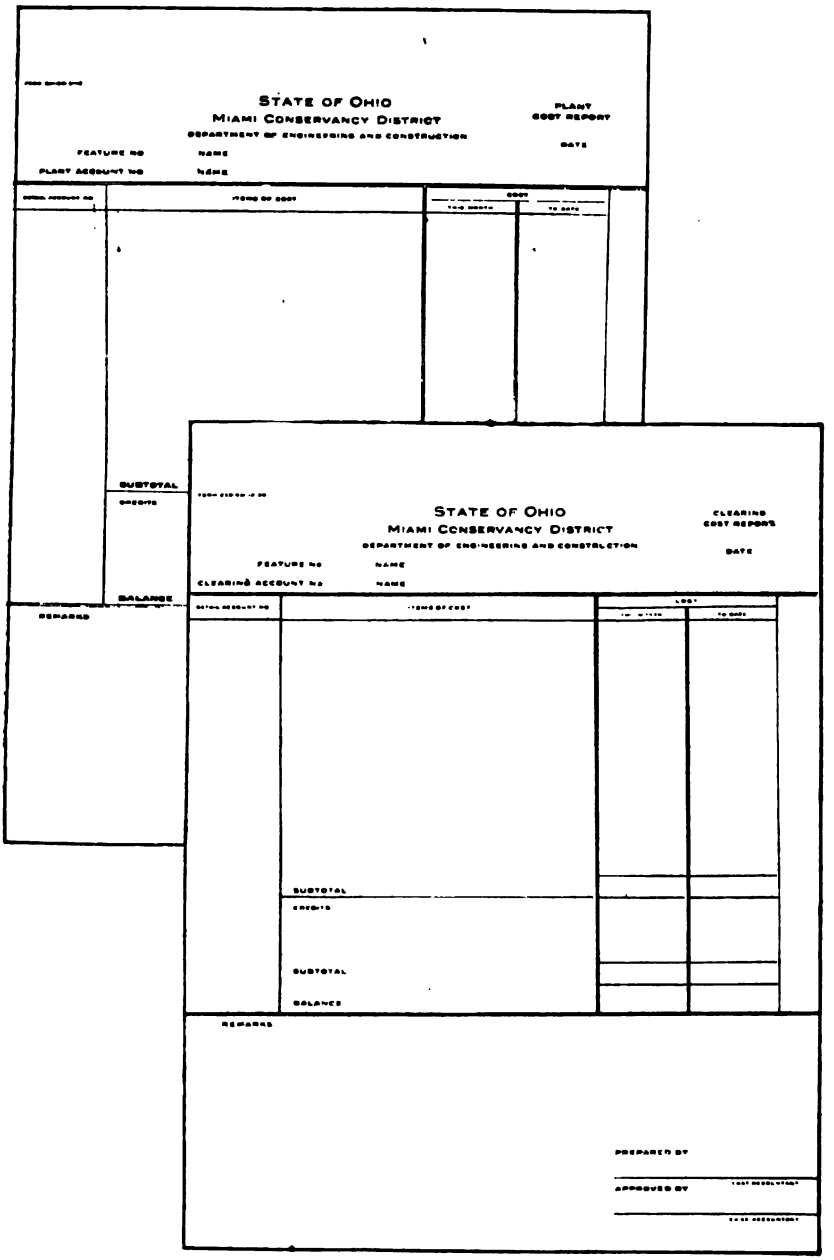


FIGURE 30—PLANT AND OPERATION OR CLEARING ACCOUNT REPORT SHEETS

STATE OF OHIO
MIAMI CONSERVANCY DISTRICT
DEPARTMENT OF ENGINEERING AND CONSTRUCTION

FINANCIAL STATEMENT

Month of 192

	Dr	Cr	Dr Balance	Cr Balance	Dr	Cr	Dr Balance	Cr Balance	Net Balance
ASSETS									
I. CASH									
II. INVENTORIES									
11 Reconstruction									
16— Warehouse Stock									
Factory —									
Sub. Total—									
17 — Inventory Depreciation									
18— Freight and Handling									
19 — Goods in Transit									
III. ACCOUNTS RECEIVABLE									
21 Personal Accounts									
25 — Unrelated Inter Dept. Transfers									
IV. ADMINISTRATIVE ADJUSTMENT									
30 Assets									
V. CONSTRUCTION COST									
31 Future Items									
32 Plant									
33 — Operation									
36— Salaries									
37 — Discounts									
38— Miscellaneous Business									
VI. 40— Maintenance									
LIABILITIES									
VII. ACCOUNTS PAYABLE									
41 — Unpaid Labor									
42 — Unpaid Purchases									
45— Unpaid Contractor Estimates									
46— Unpaid Contractor Withholds									
VIII. 47 — Income — Accident Ins.									
IX. ADMINISTRATIVE ADJUSTMENT									
50 — Liabilities									
X. INVESTMENT ACCOUNT									
61 Debentures									
62 — Transfers Received									
Less									
71 — Collections									
72 — Transfers Issued									
NET INVESTMENT									
XI. ADMINISTRATIVE ADJUSTMENT									
80 — Contra —									

Prepared by ..... Assistant Chief Accountant Approved ..... Chief Accountant

FIGURE 31—FINANCIAL STATEMENT, FORM E-214

CHAPTER XIV
BUDGET

The amount of money required for construction was based, in the first instance, upon preliminary estimates of cost under the approved Official Plan. Very careful estimates were worked out and financing was based upon these estimates as a whole.

Owing to the conditions resulting from the entry of the United States into the World War, cost exceeded the original estimates, and early in 1920 a revision of these estimates was made and additional bonds sold to finance the job to completion.

The budget system was based upon these estimates. Allotments were made to features by items and every effort made to keep the unit cost within the estimates making up the allotment. From time to time (usually at yearly intervals) revised estimates of cost to complete were prepared and allotments revised accordingly. Actual cost was compared with the estimate by monthly statements prepared for the job as a whole. This statement showed amount allotted by feature, cost to date, and balance available to complete. A more detailed analysis by items, for each feature, was also prepared at the same time, comparison being between estimated and actual unit cost.

The comparison could not be made by features on a cash basis because there was no practicable way of ascertaining the exact amount of cash required by features for a given period. The investment had already been made in plant, large stocks of materials and supplies were on hand in the warehouses, and deferred charges of various kinds were involved, all of which entered into subsequent cost but demanded no additional outlay in cash. On the other hand, a large amount of cash was required to meet accounts payable which had no relation to future cost. Budget comparison by features was therefore made on a cost basis and the cash requirements computed for the construction department as a whole.

From the total estimates of cost there were deducted:

1. Value of materials and supplies on hand.
2. Equipment depreciation included in cost estimates.
3. Accounts receivable.

To the remainder were added:

1. Accounts payable.
2. Additional investment needed for plant.
3. Reserve for accident insurance, etc.

In this manner the total amount of cash needed, based on estimates as rendered, could be approximately ascertained after making due allowance for uncollectible accounts receivable and contingencies.

The making of budget reports was not begun until 1920. Economy would probably have resulted if the system had been used from the beginning, judging by the interest it stimulated in division engineers and superintendents after it was installed.

CHAPTER XV

GENERAL EXPENSE OR OVERHEAD

Job overhead consisted of field engineering, superintendence and field accounting, depreciation, camp expense, and expense intimately connected with construction operations. All of such expenses were included in items of work through the various operating accounts as explained in the discussion of these operations (pages 28 to 32).

Another class of expense, which has been called "general expense," consisting of the headquarters office, legal and administrative offices and financing, was largely foreign to actual construction operations. It will be noted from the classification on page 23 that headquarters expense for the construction department was carried as a separate feature (Nos. 201 to 213). This contained no part of the administrative (or treasurer's) office, legal office or financing. It was the construction headquarters office only. But it was a part of construction cost and was charged to features. It would not have been practicable, however, to distribute it to items of work, so distribution was made at the close of each month by charging to a special item under each feature a certain percent of the feature cost. This was usually done in the month following that in which the cost was incurred. That is, a certain percent of the cost for May would be charged in June business and credited to a special credit account under headquarters expense. There was usually a small balance left in the headquarters' feature. The total of both debits and credits was carried forward for statistical purposes.

Much difference of opinion exists as to the best method of distributing general expense and volumes have been written about it. It is conceded that the District's method was not scientific, or even fair to all features, but no absolutely fair method was possible of application under the conditions which existed. Probably the direct labor method would have been better, but so far as this work was concerned no convincing advantage was apparent in any particular method as compared with any other one. Therefore the simplest was the best. The only reason for the distribution was to avoid misleading information as to the probable total cost to date of any one feature. Feature cost reports were subtotaled before adding this expense, and the addition called attention to the fact that general expense was being taken into consideration.

CONCLUSION

In bringing this report to a close, it will not be amiss to repeat what has been inferred before. The importance of the work of the accountant and cost keeper as an aid to the engineer can hardly be overstated. Cost keeping is the latest development in the accounting field, and from the viewpoint of the engineer, the most important. The engineer is interested in financing and bookkeeping in a general way, but in the actual carrying out of his construction program some method of cost finding is indispensable. The term accounting as used in this report embraces the broad field of which cost keeping is a part. Some deny the relationship between accounting, or bookkeeping, and cost keeping, and maintain that the cost keeping should be entirely divorced from the bookkeeping. By taking from bookkeeping some of its commercial traditions it is adaptable to the keeping of costs on construction work, as well as for manufacturing. All cost records, no matter by what term they are known—should be susceptible of control or check by the financial books, and to this extent at least the relationship should be maintained.

Information based upon the experience of the District in the engineering field is being widely utilized by the engineering profession, and it is to bring the accounting phase of the project to general attention that this report is added to the literature already published.

APPENDIX

COST PLUS VARIABLE FEE CONTRACTS

As has been said before, a very large percentage of the construction work was performed by the District's own forces. In fact, practically all of the main features of the work were so handled. There were many cases, however, where some of the smaller jobs, or some of the independent units of the large features could be handled more economically by local contractors, who had suitable equipment available and who could give their personal attention to the details of the work. So, with the District acting as the main contractor, many such jobs were "sublet," under forms of agreement best suited to the work in hand.

In general, "unit price" construction contracts are impersonal affairs, written so that the lowest bidder, no matter who he may be, can do the work, provided, of course, that he satisfies the usual requirements as to experience, financial ability, plant available, etc. A number of such sub-contracts were let by the District. These were generally for work of standard nature, such as independent sections of levee embankment, road fills, etc., where, to get the dirt into place, was the main requisite. But on some of the work the experience and reliability of the contractor were of first importance, and in some cases the work was of such special nature that a unit price bid was, of necessity, a gamble. A cost-plus-variable-fee contract was devised to meet these conditions.

This type of contract is quite different from the ordinary "cost-plus" contract, where the contractor's fee is either a fixed sum, or else a fixed percentage of the cost. The cost-plus-fixed-fee contract is objectionable, because the contractor's fee is the same regardless of whether he handles the work economically or not. The cost-plus-percentage contract is even worse, because under that form of agreement, the more the work costs the larger the fee. In the cost-plus-variable-fee contract those objections are overcome by giving the contractor the incentive to keep down costs and thus increase his fee. The more he saves for the District, the larger will be his own compensation.

The first important element in this form of contract is the "base price." This base price is a fair estimate of what the work should cost. It is obtained by multiplying estimated unit prices by actual quantities. The estimated unit prices are agreed upon at the time the contract is made, are based on labor rates in effect at that time, and are subject to modification in proportion to changes in labor rates as the work progresses. Actual quantities are obtained from monthly estimates as the work is performed. In the event that the actual cost equals the base price, the contractor's fee is set at some percentage of the base price (usually about 8% to 10%). If the work costs more than the base price, then the contractor's fee (always a percentage of base price, not of cost), is reduced by a percentage of the excess cost. The larger

the excess cost, the smaller becomes the fee. A limit (usually 3% of the base price) is set, below which the fee cannot shrink. This 3% is barely enough to cover the contractor's overhead expense. If the work is done for less than the base price, the contractor shares in the savings on a sliding scale. The largest amount that he can earn may be limited to a definite percentage, but is always less than 50% of the saving.

In short, the contractor's fee depends on the relation of actual cost to base price. The higher the cost, the smaller the fee; the lower the cost, the larger the fee. Neither he nor the District have to gamble on labor rates because an adjustment is provided for in case wages increase or decrease during the progress of the work. The contractor has a chance to earn more than his ordinary fee, but in so doing he makes a greater saving for the District. On the other hand, he is protected against "going broke" because his minimum fee is enough to cover his actual overhead expense, although under the worst condition he may get practically nothing for his own services.

Under this type of contract the contractor is really an employee in a very responsible position, and is employed and treated as such. The contractor, in turn, regards the District as his employer and partner. Both the District and the contractor strive for the common end of good work and economy, for they both profit thereby.

There are three essentials to such an agreement. The first is a good estimate of probable cost, to be used in making up the base price. An estimate either too low or too high will defeat the object desired. The second essential is mutual confidence between the two parties. The third is fair dealing on both sides during the progress of the work. If any of the essentials are lacking, the contract will prove disappointing.

A number of such sub-contracts were let by the District. The results were very satisfactory. In most of them, the work was done for less than the estimated price, to mutual profit. In every case, the work was done for substantially less than it would have cost under an old-style unit price bid.

The general provisions and detail specifications of these agreements were taken from the standard contract forms of the District, as published as Part VI of the Technical Reports. Paragraphs specifically setting forth the special provisions of this particular form of agreement were added. Such special paragraphs are illustrated by the example which follows, taken from an agreement for placing concrete revetment on the levee slopes.

TERMS:—The District shall pay and the Contractor shall accept as full compensation for work done, the actual necessary construction cost incurred by the Contractor, increased by the Contractor's fee as hereinafter specified.

CONSTRUCTION COST:—The construction cost shall include all labor, materials, supplies, and expense directly chargeable to the work whether supplied by the District or by the Contractor. There shall be included within this cost, expense of installation and dismantling of plant,

electric power, labor of timekeepers and foremen, liability insurance, agreed charge for use of equipment and other similar items which go to make up the complete job, except, however, that the following items shall not be included in the construction cost: salary and personal expense of the president of the Company, including the maintenance and operation of his personal automobile; cost of operation of the Contractor's central office; any similar expense not directly chargeable to the work; the cost of cement, reinforcing steel, expansion joint material and wire strand, which will be furnished by the District f. o. b. cars or in its warehouse at ----------; and the cost of sand and gravel which will be furnished by the District at screening plant or pit. For the determination of the actual cost, the Contractor shall submit:

(1) At the beginning of the work and at any time thereafter when a change is made, a statement of the plant and equipment which he proposes to use, including first cost and present value, said statement to be approved by the Chief Engineer of the District, or his representative, before the plant or equipment is placed in use.

(2) Each week a statement of all materials received, specifying whether such materials were purchased through the District or direct by the Contractor, and their cost in detail.

(3) Each week, payrolls of employees, teams, etc.

For the purpose of obtaining and checking payments due the Contractor under this agreement, access shall be given to the District at all times to all such books and accounts as contain a record of any and all labor supplied and expenditures made, or to be made, which are chargeable to this work. Such data, checked and supplemented by the records of the District, shall be the basis of the computation of the construction cost.

THE CONTRACTOR'S FEE:—The Contractor's fee will be determined in the following manner: The estimated unit cost of the various items of work contemplated under this agreement is as follows:

Schedule of Estimated Unit Prices

Excavation for foundations----------per cu. yd.
Constructing monolithic slab revetment----------per cu. yd.
Laying flexible slab revetment----------per block.

It is agreed that the estimated unit prices given above are based on labor conditions on the date of this agreement, and that these prices shall be increased or decreased in the same ratio that the District finds necessary to increase or decrease its wage rate for common labor.* On the date of this agreement this rate for common labor is----------per 10-hour day.

At the end of each month, a determination will be made of the total actual quantities of each item of work mentioned in this schedule completed to the end of such month. The sum of the amounts obtained by multiplying each such quantity by the corresponding unit price as named in the schedule will be termed the base price. Should the actual construction cost up to the end of such month be equal to the base price, the Contractor's fee shall be 9% of the base price. Should the actual construction cost exceed the base price, the Contractor's fee shall be decreased by 25% of the excess cost up to an excess of 10% of said base price, and by 50% of all additional excess cost above 10% of said base price, provided, however, that the Contractor's fee shall not be reduced to less than 3% of the said base price.

Should the actual construction cost be less than the said base price, the Contractor's fee shall be increased by 25% of the saving in cost below the base price up to a saving of 10% of said base price, and by 50% of all additional saving above 10% of said base price.

PURCHASE OF MATERIALS AND SUPPLIES:—All materials and supplies required by the Contractor for this work shall be ordered on regular forms provided by the District, will be furnished by the District subject to the approval of the Chief Engineer and charged to the

* The District was maintaining its own construction forces during this period, so its established rate for common labor at any time, as well as any changes in such rate, were matters of record.

work at actual cost plus 15% to cover the expense of purchasing, accounting, and overhead; except that in case of absolute necessity the Contractor may purchase direct, items costing less than five dollars ($5.00) per unit, which will be charged to the work at cost, and will be paid for by the District, on a sundry expense account if approved by the Chief Engineer or his authorized representative.

Equipment used upon the work, whether supplied by the Contractor or the District, shall be charged into the construction cost at the rate of 2% per month on the value of the equipment. This value shall be agreed upon by the Contractor and the Chief Engineer of the District as a reasonable market value of the equipment at the time it is assigned to the work. This charge for use of equipment shall begin at the time the equipment is put into use, and shall continue whether the equipment is in use or not as long as it is held on the job, or until such time as it is listed by the Contractor as available for permanent transfer to other work.

MONTHLY ESTIMATES:—Payment to the Contractor will be made on or before the 20th of each month for the work performed during the previous month, and shall cover the actual cost to the Contractor, including the agreed rate for use of his own equipment, increased by the graduated fee specified heretofore and due at the date of the estimate. The computation of the payment due will be made on the basis of total work done, and total fee earned from the beginning of the work to the date of the estimate with previous payments deducted. It is agreed that in the monthly estimates 50% only of the Contractor's fee will be included for payment until a reserve of $_____is reached. Upon the completion of the work, in full satisfaction of this agreement, a final estimate will be computed for all work done, and full payment including the portion of fee previously withheld, will be made.

Payments on account may be made each week in sufficient amount to pay for the Contractor's payroll for the preceding week.. All such payments made between any two succeeding monthly estimates shall be deducted from the amount due as shown upon such monthly estimates. Said weekly payments are subject to correction at the time of computing the monthly or final payments, should errors be found necessitating such correction.

TERMINATION OF AGREEMENT:—It is the essence of this agreement that the work provided for herein shall be carried on with the greatest efficiency and economy. To this end, it is understood that the president of the Company shall devote his attention to the work in all its details to obtain an efficient organization, a control over purchase and use of materials and supplies, and the reduction of all waste. It is the expectation of both parties in this agreement that it will remain in force until the completion of the work. However, this agreement may be terminated either by the District or by the Contractor upon ninety (90) days' notice, or may be terminated at once by the District upon the disability of the present president of the Company. In case of such termination, the plant and equipment shall be at the disposal of the District for the completion of the work, but such compensation therefor as shall be just shall be made to the Contractor.

STATE OF OHIO
THE MIAMI CONSERVANCY DISTRICT

Construction Plant, Methods and Costs

BY
CHAS. H. PAUL,
Chief Engineer

TECHNICAL REPORTS
[illegible]

DAYTON, OHIO
[illegible]

FRONTISPIECE—THE GERMANTOWN DAM AND RETARDING BASIN

STATE OF OHIO

THE MIAMI CONSERVANCY DISTRICT

Construction Plant, Methods and Costs

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Chief Engineer

TECHNICAL REPORTS

Part X

DAYTON, OHIO

1925

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THE MIAMI CONSERVANCY DISTRICT

DAYTON, OHIO

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PREFACE

This volume is the tenth and last of a series of Technical Reports issued in connection with the planning and execution of the system of flood control works built by The Miami Conservancy District.

The great flood of March, 1913, was general over the watershed of the Ohio River and its tributaries, but was particularly destructive in the Miami Valley, which comprises about 4000 square miles of gently rolling territory in southwestern Ohio. Over 360 lives were lost and more than $100,000,000 worth of property was destroyed in this section, in which are numerous towns and cities that are rather highly developed industrial centers. As a result of this destructive flood, steps were taken immediately toward the development of a plan for the protection of the entire valley against a recurrence of the disaster. The Miami Conservancy District, established in June, 1915, under the newly enacted Conservancy Act of Ohio, became the agency for securing this protection.

The plans for flood control, which were developed with more than ordinary care both on account of the size of the undertaking and because of the novelty of the project, included the building of five earth dams across the valleys of the Miami River and its tributaries, to form retarding basins, and the improvement of several miles of river channel through the towns and cities of the valley. The dams contain 8,000,000 cubic yards of earth, and over 160,000 cubic yards of concrete were placed in their outlet structures. The river channel improvements involved the excavation of 5,300,000 cubic yards, the placing of 2,600,000 cubic yards of embankment in levees, and nearly 90,000 cubic yards of concrete in walls and slope paving. The cost of the entire project, including damages and public service relocations, was about $30,000,000. The general features of the plan are outlined in the early chapters of the present volume, and the size and characteristics of the various sub-divisions of the project are given. For more detailed discussion of the plan

and of the general design of the principal structures, reference should be made to other volumes of these Technical Reports.

As practically all of the construction work was done by an organization and equipment brought together by the District for that purpose, this volume has been prepared to present a record of the methods and plant used to carry out the work, together with such cost figures as are pertinent. The results achieved have been due, in large measure, to the ingenuity, energy, and unusual ability of C. H. Locher, Construction Manager.

Following a general description of the construction plant used on the various units of the project, together with the reasons for selecting each of the more important items, is given a complete list of equipment for each unit and the cost of each item. There is included also the cost of installation and dismantling, and the total investment in plant for each job. Construction methods and results have been discussed for the more important items of work, or for any work that was of unusual character, and the costs of the various classifications of work have been set down and analyzed.

Much of the material has been obtained from the final reports and other memoranda of the Division Engineers and others in charge of the various units of the work. The arrangement and method of presentation of the material has been made so as to enable the reader to obtain complete information regarding any phase of the work without having to resort to cross references. To accomplish this, considerable repetition was necessary, but it is believed that the convenience of the reader is best served by the method used.

The cost records given include, except where specifically noted, all proper charges to the work, such as distribution of shop operation, garage and warehouse charges, and other similar costs which are not direct charges against any one item of work. It should be noted also that the work was carried on during and immediately after the World War, and the effects of labor and material shortages, as well as the intangible effects of war conditions, were naturally reflected in the costs, which are somewhat higher than they would have been under normal conditions. The very nature of the work called for its early completion notwithstanding the adverse conditions. In spite of this, because of the efficiency and splendid loyalty of the organization, these costs are not far out of line for present day conditions.

Throughout this volume, cost and performance records have been selected to illustrate average, all-the-year-around results, rather than unusual records. Enough "maximum performance" and "least cost" data have been given to show what can be done under favorable conditions, but it is desired to emphasize the fact that it is the average every-day performance that counts in the long run, and estimates on new work should be based on such performance rather than on unusually favorable past records.

So many members of the engineering and construction staff of the District have contributed, either directly or indirectly, to the material in this volume, that it is impracticable to undertake to mention them individually. The author's job has been largely one of selecting, adapting, analyzing and arranging available material. Special acknowledgment is made to C. S. Bennett, Engineer, Miami Conservancy District, who not only assisted materially in preparing and editing much of the manuscript, but also has had full charge of the mechanical details of making the book.

Chas. H. Paul.

Dayton, Ohio,
June, 1925.

THE TECHNICAL REPORTS

More than usual care attended the preparation of the plans for the thirty-four million dollar flood control works for the Miami Valley in Ohio. Some of the problems required study and research work of an exhaustive character. The Technical Reports are issued by the Miami Conservancy District in response to the demand for information concerning the work and the results of the investigations conducted by the engineering staff. The data published is in some respects entirely new, and much of it is not available in convenient form elsewhere.

The reports, which are published in volumes, called Parts, size 6 x 9 inches, bound in paper covers, are as follows:

PART I—THE MIAMI VALLEY AND THE 1913 FLOOD

By Arthur E. Morgan,

Chief Engineer of The Miami Conservancy District.

128 pages, 6 x 9 inches, 44 illustrations, paper covers; price, 75 cents net, postpaid.

This volume is introductory to the series, but is not in itself, of a technical nature. It contains a description of the flood of March, 1913, which caused great damage in the Miami River Valley, and as a result of which the flood control works now being built were undertaken.

PART II—HISTORY OF THE MIAMI FLOOD CONTROL PROJECT

By C. A. Bock,

Division Engineer of The Miami Conservancy District.

194 pages, 6 x 9 inches, 41 illustrations, paper covers; price 50 cents net, postpaid.

This is a recital of events of interest in the history of the flood prevention movement in the Miami Valley. Its chief value lies in the fact that it describes the steps necessary and the difficulties usually encountered in initiating and carrying out a great public engineering enterprise of this nature.

PART III—HYDRAULIC JUMP AND BACKWATER CURVES

By Sherman M. Woodward, R. M. Riegel and J. C. Beebe.

111 pages, 6 x 9 inches, 88 illustrations, 12 tables, paper covers; price 50 cents net, postpaid.

The first part of the book, entitled "Theory of the Hydraulic Jump and Backwater Curves," sets forth the theory of these complex phenomena, and is written by S. M. Woodward, Professor of Hydraulics and Mechanics at Iowa University, and Consulting Engineer for The Miami Conservancy District.

PART IV—CALCULATION OF FLOW IN OPEN CHANNELS

By Ivan E. Houk,

Hydrographer of The Miami Conservancy District.

283 pages, 6 x 9 inches, 79 illustrations, 48 tables, paper covers; price 75 cents net, postpaid.

This report describes how the maximum flow of the Miami River and its tributaries during the disastrous flood of March, 1913, was determined by subsequent measurements of the river channels and flood elevations. A large part of the volume is devoted to a discussion of different formulas, many of them of foreign origin, used in computing stream flow.

PART V—STORM RAINFALL OF THE EASTERN UNITED STATES

By the Engineering Staff of The Miami Conservancy District.

310 pages, 6 x 9 inches, 114 illustrations, 11 tables and appendix, paper covers; price 75 cents net, postpaid.

This book sets forth the results and investigations made to reach safe conclusions as to the probable size and frequency of severe floods in the Miami River. Its most noteworthy feature is a study of the relations between the areas covered by different depths of rainfall and the duration of such rainfall for each of the 33 largest storms on record in the eastern half of the United States.

PART VI—CONTRACT FORMS AND SPECIFICATIONS

By the Engineering Staff of The Miami Conservancy District.

192 pages, 6 x 9 inches, 3 diagrams, paper covers; price 50 cents net, postpaid.

This volume has been issued to meet the continued demand for copies of the standard specifications and contract forms in use by The Miami Conservancy District, in the construction of the large earth dams and the protection works along the river channels.

ATLAS OF SELECTED CONTRACT AND INFORMATION DRAWINGS TO ACCOMPANY PART VI.

10 pages of text, 15 x 11 inches, 139 plates, paper covers; price $1.50 net, postpaid.

The Atlas contains 139 representative selections of plans made preparatory to the letting of contracts.

PART VII—HYDRAULICS OF THE MIAMI FLOOD CONTROL PROJECT

By Sherman M. Woodward

Consulting Engineer for The Miami Conservancy District.

344 pages, 6 x 9 inches, 126 illustrations, 37 tables, index, paper covers; price $1.00 net, postpaid.

This volume makes an orderly presentation of the methods followed in determining the type of the flood control plans and of the methods of

working out the application of the plan. If such a presentation of the subject had been available when this project was undertaken, its use would have saved a very large amount of time and effort.

PART VIII—RAINFALL AND RUNOFF IN THE MIAMI VALLEY

By Ivan E. Houk,

Hydrographer of The Miami Conservancy District.

About 220 pages, 6 x 9 inches, 51 illustrations, 40 tables, paper covers; price, 75 cents net, postpaid.

This report is a record of the results of the studies of the Miami Conservancy District on rainfall and runoff. A considerable part of the volume deals with experiments with both natural and artificial rainfalls.

PART IX—ACCOUNTING AND COST KEEPING OF THE DEPARTMENT OF ENGINEERING AND CONSTRUCTION

By F. L. Cavis,

Chief Accountant of the Miami Conservancy District.

112 pages, 6 x 9 inches, 31 illustrations, paper covers; price 75 cents net, postpaid.

The size and unique character of the enterprise, and the fact that it was public work and carried out by the District's own forces, necessitated the development of an unusually complete accounting system. This report presents an accounting system that has had unworkable features taken out of it under the pressure of actual operations.

PART X—CONSTRUCTION PLANT, METHODS AND COSTS

By Chas. H. Paul,

Chief Engineer of The Miami Conservancy District.

432 pages, 6 x 9 inches, 130 illustrations, 31 tables, paper covers; price, $2.00 net, postpaid.

Checks should be made in favor of The Miami Conservancy District, and all orders and communications should be addressed to:

THE MIAMI CONSERVANCY DISTRICT,
Dayton, Ohio.

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CHAPTER I.—INTRODUCTION

General Description of Project

The flood of March, 1913, was the greatest ever recorded in the Miami Valley. Large sections of the business and residential districts of Dayton and Hamilton were inundated to depths of 10 to 12 feet. Other cities and towns in the valley sustained severe damage. The property loss was estimated at $100,000,000, and about 360 deaths were directly traceable to this unprecedented flood.

Relief committees were organized in the different cities immediately after the flood, and plans for the prevention of future floods began to be formulated. Each community worked independently at first, the thought being that the desired results could be accomplished by channel enlargement, and that each community could work out its own protection plans. Soon after systematic engineering study was begun, however, it became apparent, first, that channel enlargement alone, to the extent required, was not practicable, and, second, that the people of one city alone could not adequately protect themselves, but that it was a job for the people of the whole valley to undertake as a unit. A combination of channel improvement and retarding basins was the final solution of the problem.

The river system of the Miami Valley consists of three main branches, coming together at the City of Dayton. These are the Stillwater, the Miami and the Mad Rivers which flow on south under the name of the Miami. Wolf Creek joins the Miami also at Dayton, just below the junction of the three main branches. At the upper end of the valley Loramie Creek joins the Miami branch just above Piqua; Twin Creek joins the main river between Franklin and Middletown, and Four Mile Creek comes in just above Hamilton. The drainage area above Hamilton is 3600 square miles and above Dayton it is 2600 square miles. Fig. 1, a map of the valley, shows the course of the river and its main tributaries.

The flood prevention plan for this valley, which was worked out and carried through to completion by an organization known as The Miami Conservancy District, provides for channel improvement and levee systems through all the cities and towns affected by floods, to the extent that is economically feasible,

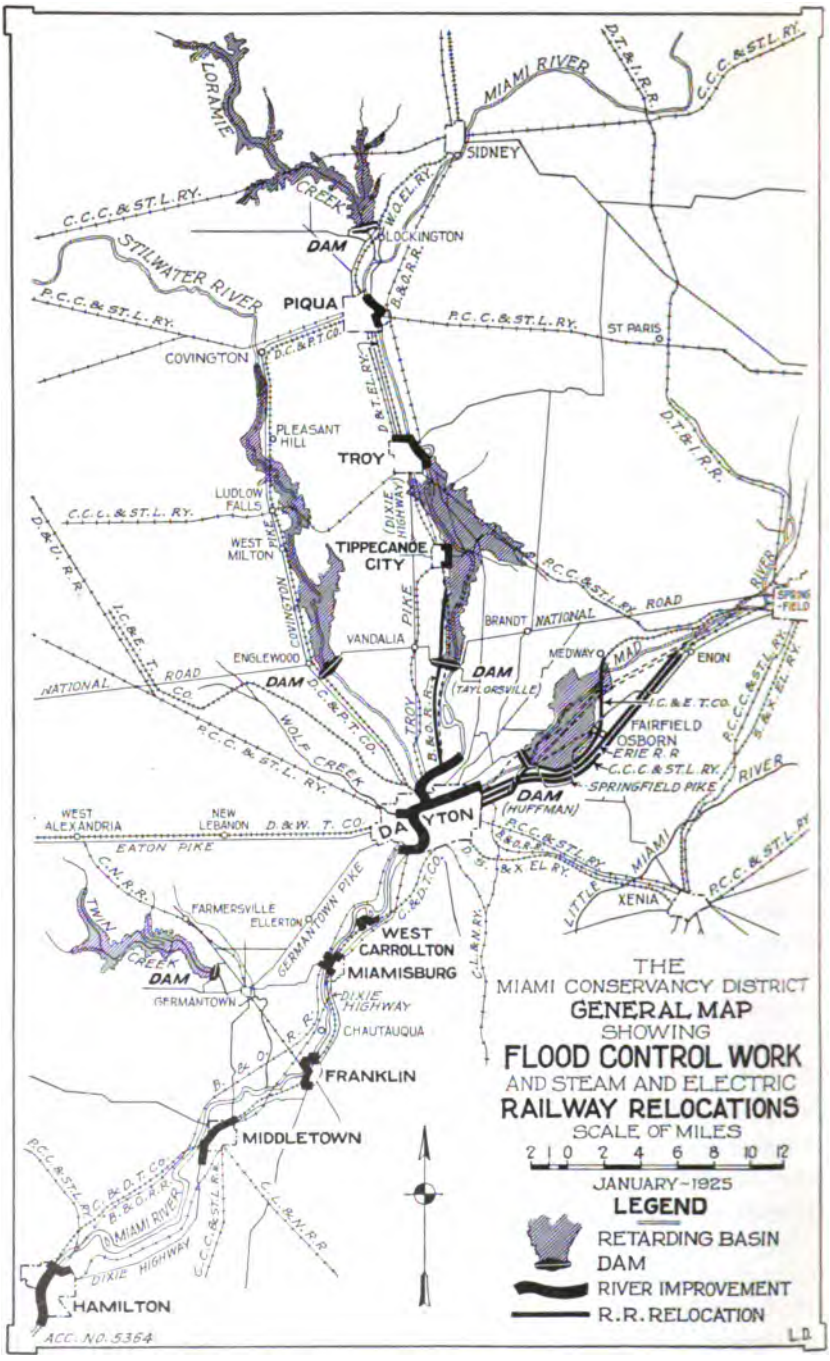


FIG. 1—GENERAL MAP

supplemented by retarding basins which will hold back the crest of the flood by restricting the flow through the dams, and backing up the surplus water temporarily in the basins. Fig. 1 shows the location of the various principal features of the work. Extensive channel improvement is provided at Piqua, Troy, Dayton, West Carrollton, Miamisburg, Franklin, Middletown and Hamilton. Tippecanoe City is protected by a levee, from backwater in the Taylorsville basin. Dams which form retarding basins are located at Lockington on Loramie Creek, at Englewood on the Stillwater River, at Taylorsville on the Miami, at Huffman on the Mad River and at Germantown on Twin Creek. It was necessary also to relocate more than 50 miles of railroad lines in order to get them out of the way of the dams and the retarding basins. These railroad relocations are also indicated on the map, Fig. 1.

The channel improvement work consisted of deepening, widening, or straightening the river channels, involving excavation to the extent of 5,300,000 cu. yds.; building or raising levees, requiring a large amount of embankment work; paving slopes; lowering water mains and other structures crossing the rivers; extending or modifying sewer outlets; placing flood gates on sewers where there was a possibility of flood waters backing through into the cities; street relocation and numerous other items of work of a similar nature.

The five dams forming the retarding basins were all built of earth by the hydraulic fill method. The smallest of these dams contains 800,000 cu. yds. of embankment and the largest 3,600,000 cu. yds. In maximum height they range from 73 to 125 ft. All of the dams have massive concrete outlet and spillway structures which required considerable quantities of high class concrete construction. Materials both for the hydraulic fill embankments and for the concrete aggregates were obtained in the immediate vicinities of the damsites.

Incidentally, in connection with this work, nine new reinforced concrete highway bridges were built, several existing highway bridges were raised or relocated, and one steel truss bridge over the Stillwater River was dismantled, moved, and reerected at another site about a mile away. Many highways crossing the retarding basins were raised or relocated.

The principal quantities of work involved in the construction heretofore mentioned are shown under appropriate head-

Table I.—Quantities of Work, by Features

Feature	Excavation	Embankment	Concrete	Steel
Channel Improvement				
	Cu. Yds.	Cu. Yds.	Cu. Yds.	Tons
Piqua	260,315	338,252	1,298	2.40
Troy	343,822	180,664	5,494	132.43
Tippecanoe City	43,773	171,776	4,161	33.95
Dayton	2,195,779	869,537	35,715	570.72
West Carrollton	2,962	34,697	286	.....
Miamisburg	26,533	232,298	4,601	46.38
Franklin	107,330	143,026	4,638	120.34
Middletown	84,174	186,602	1,345	19.03
Hamilton	2,265,716	407,782	31,615	885.64
Sub-totals	5,330,404	2,564,634	89,153	1,810.89
Dams				
Lockington	267,691	1,022,868	32,062	29.30
Englewood	339,225	3,642,600	27,391	147.16
Taylorsville	818,390	1,301,262	47,934	80.26
Huffman	318,515	1,402,490	37,822	188.23
Germantown	189,554	844,577	17,381	46.72
Sub-totals	1,933,375	8,213,797	162,590	491.67
Totals	7,263,779	10,778,431	251,743	2,302.56

ings in Table I. It will be seen that in many instances these quantities are very large. The total cost of the project was about $30,000,000.

At the time preliminary plans were being worked out it was expected that the construction work would be done by contract, and the principal divisions of work were advertised for bids in the fall of 1917. The United States in the meantime had entered the World War. Labor and industrial conditions had become acute, and it was realized that in justice to himself no reliable contractor could make a reasonable bid on jobs such as this, which would require 4 or 5 years to complete. Bids were opened in November, 1917, and with the exception of one bid for one section of channel improvement, all of them were either unreasonably high or were irregular. It was then decided that the District itself should organize its own construction force, purchase its own equipment, and go ahead with the work on a force account basis. Thus it came about that The Miami Conservancy District built up a large construction organization, acquired a large and complete construction plant, and undertook

the construction operations, which it is the purpose of this report to describe.

Another glance at the map, Fig. I, will show that adequate railroad facilities existed to serve all of the channel improvement sections of the project. In addition, most of the cities and towns are served by traction lines which carry freight as well as passengers. Of the five dams, two, Taylorsville and Huffman, were located on the main lines of railroads; one, Englewood, was within less than a mile of an existing railroad line and could be served with a connection therefrom; and one, Lockington, was within easy reach of a traction line which in turn maintained interchange with a railway system not far away. The fifth dam, Germantown, was about three miles from the nearest railroad with rough, hilly country between, so that it was not feasible to build a railroad connection in to it, but it was necessary to haul all supplies, materials and equipment for that work over about three miles of hilly country roads.

The main highways within the District were generally in fair condition, although only a portion of them were hard surfaced roads. It was possible, however, to get at any point of the District's operations, at any time of year, over a road in at least fair condition. A paving program was in operation at the beginning of the work, and before the District's construction operations were finished there were paved roads leading to almost every point where construction was being carried on.

Here, then, was a big job, split up into large subdivisions, each practically a job in itself. These separate units each required its own organization and construction plant, but they were close enough together, and transportation facilities were such, that they could be served from a central warehouse and a central repair shop, with only such warehouse and shop facilities on the job as were required to take care of immediate or emergency needs. Transfer of equipment from job to job was practicable for the same reasons, and standardization of equipment was desirable in-so-far as possible, because of the advantage of keeping down the necessary stock of repair parts.

Most of the channel improvement jobs were within a few miles of one or more other jobs. Each of the dams was within ten miles or less of some other job. It was only about 75

miles from Lockington Dam to Hamilton, the two jobs at the extreme north and south limits of the construction work. Dayton, where the headquarters was established, was about at the geographical center of the work, and less than 3 hours' distance by truck from the jobs farthest away.

Special telephone lines were run out from headquarters to each of the important jobs, and telephone stations were scattered out on each job at economical points.

Power for Construction Plant

Before the purchase of equipment was undertaken it was necessary to decide the source of power to be used on the construction plant. Except for small units, gasoline power was not considered, neither were oil-fueled boilers nor internal combustion oil engines, for the reason that they were not in common use, and even if found practicable, they would be hard to obtain under the conditions existing at that time. The choice quickly narrowed down to steam or electric power.

The work was to be done at many different points, some of them widely separated. The work would be heavy and would require energy in large quantities totaling in the neighborhood of 10,000 horse power. The applications of the power would be of great variety. There would be large excavating machines requiring several hundred horse power each. There would be pumps for various purposes and of a great variety of sizes, from small unwatering pumps up to dredge pumps requiring as much as 500 horse power each. Power would be needed for gravel washers, concrete mixers, air compressors, locomotives, derricks, drills, shop equipment, lighting and various other uses. For some of the equipment such as locomotives, which would be used largely for hauling material from borrow pits extending over wide areas, steam power was the only suitable selection under prevailing conditions. For most of the other uses, however, either steam or electric power would be generally satisfactory.

The water in this vicinity is very high in lime content, and under the best of conditions and with the best of attention gives much trouble in boilers. Much of the equipment must of necessity be located at more or less inaccessible points to which the transportation of coal would, in many cases, be difficult and expensive. The industrial difficulties of mining and transport-

ing coal for main supplies had already begun to develop, or could be foreseen at the time this work was being organized. So in spite of the many advantages of steam as a source of power, these disadvantages loomed up as real problems in connection with the selection of construction plant.

On the other hand, electric power for the operation of excavating machines, derricks, and other equipment of that type, as well as the more common uses on pumps, mixers, etc., had come into common use and had already been proven to be fairly satisfactory. While this use of electric power was not by any means as common on construction equipment at that time as it is today, still it had passed the experimental stage and was looked upon without misgivings by those in charge of the work. It happened that the Miami Valley was well supplied with electric power. The Dayton Power and Light Company, which supplied a territory including practically all of the District work, except Hamilton and Middletown, was just completing a modern 35,000-H.P. central plant at Dayton. This, together with the other plants of the Company then in operation, assured a very dependable supply. At Hamilton, the facilities of the Hamilton Service Company were sufficient to care for the needs of the District at that point. All of this was 3-phase, 60-cycle, alternating current at 33,000 volts. At some of the dams where electric power was not at the time immediately available, the estimated requirements were sufficient to justify the extension of transmission lines to these various points, and because of the large requirements over a period of 3 to 5 years, electric power at all of the points where work was to be carried on could be assured at reasonable rates. This really meant that either steam or electric power could be used to advantage at practically every point where the major operations of the District would be carried on, and as far as preliminary estimates were concerned, there was little difference to be figured in the cost per horse power at the point of actual use, between steam and electricity.

It was the consensus of opinion that there would be certain economies of maintenance and operation in favor of electric power, that the supply would be fully as dependable as that of coal, and that the difficulties of maintaining a constant coal supply would be transferred to the Power Company which was better equipped to handle such matters and was well supplied

with storage facilities. It was certain that electric power would be cleaner, more convenient, and more comfortable for the operators, and that it would do away with water troubles in boilers. While the installation of a distribution system on each job had to be considered, a part of which would need to be more or less flexible in order to serve the moving equipment, like excavating machines, this was offset to a large extent by the saving in track system, roadways, etc., that would be necessary for coal distribution, as well as pumps, pipe lines, etc., for water supply to boilers. Under these conditions it was determined to use electric power in-so-far as practicable, realizing at the same time that steam power would of necessity be used in many cases. When the actual procuring of equipment was undertaken, it developed that in many cases where electric driven equipment was preferred, it could not be obtained except after unreasonable and unwarranted delay, and therefore in many cases steam driven units were procured as the only thing available under the circumstances. This was especially true in the purchase of dragline excavators. Twenty-one machines of various sizes were required. It would have been preferred to have all of the larger of these machines electrically operated, but there were not that many electric draglines to be obtained at that time, and consequently steam machines of the same capacity, which were available, were accepted as second choice. Incidentally this resulted in opportunities, during the progress of work, to make direct comparisons between the operation of steam and electric dragline machines on the same kind of work. Such comparisons will be taken up in more detail later. It might be said here however, that the results of those observations justified the early decision as to the choice between steam and electric power.

During the construction period a total of 57,000,000 kilowatt hours of electric power was used on the project as a whole, at an average cost of 1.8c per kilowatt hour at the point of distribution on the job. Notwithstanding the extensive use of electric power, still there was a total coal consumption of 67,000 tons, the cost of which averaged $4.48 per ton at the distributing point on the job.

The results of this extensive use of electric power fully justified the early decision in its favor. The supply was constantly reliable, due in no small measure to the excellent equipment

and efficient service of the power companies. There was no occasion during the whole period of the work when any serious delay occurred because of the failure of the power supply. There were occasional brief interruptions of course, but the system was so laid out that trouble at one point could promptly be localized and not affect other operations. Taken as a whole, the loss of time due to interruptions of electric power service was so small as to be negligible. On the other hand, the coal situation, both as to supply at the mines and transportation, got to be very acute several times during the period of construction. On several different occasions it was with the utmost difficulty that the amount of coal absolutely needed to keep the work going, was secured. Often it had to be transferred at considerable expense from one job to another, and frequently emergency purchases had to be made at excessive prices. Had steam equipment been used exclusively on the work, it would have been exceedingly difficult, if not impossible, to avoid serious delays on account of coal or car shortage. In the matter of maintenance and operation, one resident electrician on each job, with a little help here and there occasionally to meet a special situation, was sufficient to keep the distribution system in order and attend to such extensions or relocations as were necessary. Time and time again the convenience of the electric supply was demonstrated, and the electric machines were always preferred by the operators because of cleanness and comfort as compared with steam machines. Moreover, wherever it was possible to make a direct or indirect comparison as regards cost of steam or electric power, the results were always in favor of electric power.

Construction Organization

It has been explained that the original intention was to have this work done by contract, and the plans and specifications had been prepared for that purpose. War conditions made it inexpedient to handle the work in that way, and therefore the District organized its own construction forces and purchased its own equipment. The general plan of organization for construction was to maintain as nearly as practicable the relationship between engineering and construction forces that would have existed on contract work. A Construction Manager was selected who took his place in the Chief Engineer's organization, handling especially the duties which naturally would have fallen

to a general contractor on the job. On each main feature of the work was a Division Engineer with authority such as he would have had on a contract job, and a Superintendent who had direct charge of the construction forces. The superintendents reported through the Construction Manager, and the Division Engineers direct, to the Chief Engineer's office. Clerical forces served both engineering and construction organizations without overlapping or duplication of work. The specifications which had been prepared in anticipation of letting contracts, were followed with such modifications as this form of organization demanded. Although responsibility for the quality of work, efficiency of methods, rate of progress and costs were allocated much as they would be on contract work, there was a sense of co-partnership between engineering and construction forces which made for better results, more rapid progress, and lower costs than would have been possible on an ordinary contract job. In spite of the fact that the job as a whole was carried out on a force account basis, there were a number of small jobs,—isolated, or requiring special equipment, or special organization, or jobs that local men were especially fitted to handle, like team work,—which were sublet to local contractors. The supervision of these jobs then became a part of the duties of the regular District organization.

Repair Shops

A blacksmith shop, containing also a limited amount of machine shop equipment, was established on every important feature of the work, namely at each dam and at each locality where extensive channel improvement work was being undertaken. At Dayton, where the general headquarters of the work was located, a more elaborate repair shop was installed. Adjacent to it a central warehouse was maintained, from which supplies and materials were furnished to the various features, and also a garage and auto repair shop, where motor trucks and automobiles for general use were put up and cared for. It was the idea that the local shops would attend to minor repairs and such emergency cases as required immediate attention by the local forces, but that the more extensive repairs should be handled at the main shop. With truck service between Dayton and any point on the work, it was possible to bring into the main shop many items of repairs which otherwise could not

have been so handled. There was considerable economy, both in time and money, in having as many of the larger jobs as possible handled at the central shop where an adequate force of skilled men could more easily be maintained economically. The work of these repair shops consists largely of rush repairs, wherein speed, and not necessarily lowest cost, is the main requirement. When a machine is shut down for want of repairs, there is usually nothing more expensive than allowing that machine to remain out of service, and any reasonable cost connected with an emergency repair job is economical if it saves time in the operation of the machine. A large stock of repair parts for the various machines was kept on hand, so that almost any emergency could be met with only a light loss of time, and no pains were spared to rush through these emergency repair jobs with the utmost possible speed. A rather complete system of plant inspection was also maintained, often with the aid of the skilled men from the main shop, but usually by a high grade mechanic who was kept on each job principally for that purpose. Whenever a breakdown could be deferred as a result of periodical inspection, so much time was saved, as there was time between shifts and on Sundays when many of such repairs might be taken care of.

The central repair shop was usually supplied with a quantity of routine work which kept an adequate force of men constantly employed. Repairing extra buckets, overhauling machines temporarily out of use, building derricks, overhauling extra boilers, and jobs of all sorts too numerous to mention, were constantly being run through the shop, but were always set aside when necessary so that preference could be given to emergency repairs.

Every job, whether emergency or not, which came through the shop, was handled on a shop order, copy of which is shown as Fig. 2. If the work was for one of the features, the cost was charged direct to that feature when the repaired part was returned. If it was for general stock, it was charged to the warehouse stock account and later issued to a job through the warehouse, and then charged to the job.

Excavating Equipment

Reference again to Table I, will show that excavating equipment was one of the principal items of equipment to be procured, the combination of excavation and embankment requir-

MACHINE SHOP COPY

STATE OF OHIO
The Miami Conservancy District
Department of Engineering and Construction
SHOP ORDER
№ 14984

REQUISITIONER'S COPY

STATE OF OHIO
The Miami Conservancy District
Department of Engineering and Construction
SHOP ORDER
№ 14984

ACCOUNTING DIVISION COPY

STATE OF OHIO
The Miami Conservancy District
Department of Engineering and Construction
SHOP ORDER
№ 14984

TO THE FOREMAN: Date . . . . . 19 . .

You are requested to perform the following described work:

. . . . . .

. . . . . .

. . . . . .

. . . . . .

. . . . . .

. . . . . .

. . . . . And charge to account No. . . . . .

Deliver the completed order to . . . . . .

At . . . . . on or before 19

Signed . . . . . .
Superintendent

THIS SPACE FOR USE OF ACCOUNTING DIVISION

ACCOUNT NO. AMOUNT

. . . . . .

200 S.G.L. . . . . Cr. . . . . .
200 S.G.L. . . . . Cr. . . . . .
200 S.G.L. . . . . Cr. . . . . .
200-055 B Dr. Total . . . . . .

Completed . . . . . 19 . . .

Delivered . . . . . 19 . . .

Signed . . . . . .
Shop Foreman.

First two lines to be filled in by Shop Clerk

Material Cost . . . . . .
Labor Cost . . . . . .
Total Labor and Material . . . . . .
Overhead . . . . % . . . . . .
Total Cost . . . . . .

FIG. 2—SHOP ORDER FORM

ing a total of about 18,000,000 cubic yards of material to be moved. The main items of excavation were as follows: excavation for outlet structures at the dams; excavation for cut-off channels and for deepening or widening river channels; excavation for cut-off trenches at the dams; excavation for highway work; excavation for retaining walls, bridge piers, and flood gate structures; excavation in borrow pits for levees; excavation in borrow pits for dam embankments.

In many of these cases it was necessary to take a large amount of the material from below water level. The excavations for the outlet and spillway structures at the dams were all at least partially in rock, and could best be handled in trenches cut to neat lines, so that the concrete walls could be laid directly against the sides of the excavations. In some cases portions at least of these excavations were very deep, not only below water level, but below general ground level. A large portion of the material had to be loaded into cars, or some other means of conveyance, and hauled away for some distance.

There was considerable discussion as to the best type of excavating equipment to handle these various jobs. At the outlet and spillway structures for the dams, the question was largely between steam shovels and dragline excavators. On portions of the channel improvement work and at the borrow pits for embankments for dams and levees, it lay between dragline machines, dipper dredges, and dredge pumps.

It was at the outlet structures for the dams that the deepest of the excavation occurred, and a large portion of it was in rock and below water level. Under such conditions the layout for steam shovel work presented many difficulties. In some places the trench would not be wide enough for both shovel and track system without excavating to extra width. There would be constant danger of flooding the pit and putting the whole outfit out of commission for considerable periods of time. The question of track grades to get out of the pit was very serious, so much so that it could be seen that this would be a serious handicap to steam shovel operations.

While the dragline excavator had been in common use for 10 or 15 years before this work was undertaken, there was some question as to its efficiency in rock excavation, and in loading material into cars. While there were records of certain jobs here and there, where such work had been handled successfully

by dragline excavators, such examples were not by any means numerous. The advantages of the dragline machine were so great in other particulars, that is, in digging below water, loading at an elevation far above the floor of the pit, long reach, flexibility of operation, etc., that it was selected as against the steam shovel for all of the outlet and spillway excavations.

On most of the large channel improvement jobs and at all of the borrow pit excavations for the dam embankments, it was evident that steam shovels would not be suitable, as so much of the material had to come from below water level over large areas. Comparative estimates between dipper dredge and dragline excavators for the borrow pit excavations indicated little choice as between the two, but one great advantage of the dragline machine in that case was its higher salvage value after the job was finished. Suction dredges and simpler dredge pump installations were also considered for that work, but the long haul would require several booster pumps in order to get the material into final place in the dam. The dragline machine seemed to have the advantage over all other types under consideration, and therefore it was selected for most of the borrow pit excavations at the dams.

On the channel improvement work the considerations were much the same as between dragline machines, dipper dredges and suction dredges. In this case however, suction dredges were counted out, if for no other reason than because of the character of the material and the shallow cutting. Dipper dredges could not have been used to advantage without excavating to extra depths, or obtaining flotation by other expensive methods, and the long reach of the dragline was a great advantage in passing material from channel excavation over into levee embankments or waste banks. It finally developed therefore that the dragline machine was selected for practically all of the excavation operations of the District. In addition to the 21 machines purchased by the District there were 3 or 4 others in use by subcontractors before the job was finished.

The dragline excavator has occupied such a prominent place in the construction operations of the District that it has been given a special chapter in this report. (See Chapter II). Steam shovels were used on most of the railroad relocation work, which, however, was handled almost entirely by contract.

At two of the dams, Taylorsville and Lockington, the borrow pit material lay almost entirely above the elevation of ground

water. It later developed at Germantown and Huffman that additional material could be obtained economically from side hill borrow pits. Here were favorable conditions for the operation of hydraulic giants in tearing out the material from its position in the borrow pits and sluicing it to sumps, where it was picked up by dredge pumps and pumped into place in the embankment. Consequently, hydraulic giants as excavating equipment were used extensively at Taylorsville and Lockington dams, and to a more limited extent at Germantown and Huffman.

Concreting Equipment

At each of the dams a gravel washing and screening plant was erected for furnishing concrete stock for the outlet and spillway structures. These plants were standardized and all parts were interchangeable. Suitable gravel and sand for concrete was found near each site. The material was hauled to the plant in dump cars and dumped into a hopper at track level, from which a belt conveyor took it to the top of the plant to the screens. Wash water was delivered to the screens by a small centrifugal pump. The material was washed and separated into three sizes, sand, fine gravel, ranging in size from ¼-in. to 1½-in., and coarse gravel graded from 1½ to 3-in. in size. Each of these sizes passed by gravity into separate storage bins. Over-size material could either be wasted or diverted to the coarse gravel bin. In the heavy walls, it was the practice to use over-size up to about 6-in. cobbles.

A 1-yd. mixer was placed in such position that the material from each of the three bins could be fed to the charging hopper by gravity, through adjustable measuring boxes. Cement was stored in a shed near-by, and was brought to the mixing platform on small warehouse trucks as needed. The mixer discharged into side-dump cars, or bottom-dump buckets on trucks, which were hauled to the work by 3-ton narrow gauge gasoline locomotives. At Germantown and Englewood, where the long conduits were built, most of the concrete was chuted into place by gravity. At the other dams, and at the spillway structures at Germantown and Englewood, the concrete was placed by derricks. Collapsible sectional forms were used for the conduit construction at Germantown and Englewood. In building the large retaining walls at the other structures, movable panel

forms were generally used. The concrete mix varied from about 1:1¾:3½ in conduit linings to about 1:3¼:6½ in the bodies of the heavy retaining walls, the change in the mix, within the limits required, being accomplished by changing the number of sacks of cement per batch. The two sizes of gravel were re-mixed in the batch in such proportion as to use up practically all of each size, and this ratio between fine and coarse gravel was fixed to suit the conditions in the borrow pit. In other words, no attempt was made to establish a theoretical combination of the fine and coarse gravel, which would have meant a considerable waste of one size or the other from time to time. There was an advantage, however, in having some fine and some coarse gravel in each batch, as this guaranteed more uniform batches than can be obtained from one bin containing both grades of gravel, and was considered sufficient justification for the separation of the gravel into two grades. The sand was carefully proportioned in the batch. So was the water.

On the channel improvement jobs the concrete work was more scattered, and therefore required equipment which could be more readily moved from place to place. Half-yard steam operated mixers, and one-sack gasoline mixers on trucks were used extensively on these channel improvement jobs. In most cases gravel and sand was obtained locally from commercial plants or dealers, except that at Dayton there was work enough to justify the erection of a screening and washing plant similar to those used at the dams. This plant turned out more material than was required for Conservancy work, and the surplus was disposed of through a cooperative agreement with a local dealer. This was simply a gravel screening and washing plant, the output being hauled to the various jobs as needed and run through the mixing plants at these jobs. At Hamilton also a small commercial washing and screening plant was erected at two or three different points for parts of the work, but afterwards the supply there was obtained more economically from a local gravel company.

Hydraulic Fill Equipment

Early studies indicated that the hydraulic fill method was the best and most economical way of placing the embankment material in the dam. At each of the damsites the material was

pumped to the dams by 15" dredge pumps. In two cases, as has already been mentioned, the borrow pit material was broken up by hydraulic giants and sluiced to sumps from which it was pumped into place. At the other three dams the bulk of the borrow pit material was excavated by dragline machines, loaded into 12-cu. yd. dump cars, hauled to the dams by 40-ton standard gage dinkey locomotives, and dumped into shallow bins or "hog-boxes." At the hog-boxes hydraulic giants were used to wash the material to the dredge pumps. Several different makes of dredge pumps were used at the beginning of the work, for the reason that because of war conditions it was impossible to secure enough pumps of any one make to standardize on this equipment. Later, however, when conditions improved, this pumping equipment was standardized on a pump of special design worked out by the Conservancy engineers in connection with one of the standard dredge pump manufacturers. It was discovered early in the game that cast iron and semi-steel parts were not durable enough for the materials which had to be handled by this equipment, and therefore manganese steel shells, and impellers equipped with removable shoes of the same material, were used almost exclusively except during the early stages of the work. Experiments were also made with shells of white iron cast locally, and this material appeared to have great possibilities for such use. It was not perfected however, for use either in impellers or shoes.

Special electric-welded dredge pipe of high carbon steel was used in the discharge lines from the dredge pumps. This gave very satisfactory results, and had a life three or four times as long as that of standard riveted dredge pipe of mild steel which was the commercial product in general use for such work up to that time.

Standard water pumps were used for low pressure sluicing water and for high pressure service to hydraulic giants.

Transportation Equipment

It has been noted heretofore that a large percentage of the excavated material, and of that obtained from borrow pits, required transportation to embankments or spoil banks. Standard gage 12-cu.yd. air dump cars were used for this purpose, about 125 of these being required at the time of maximum operations. Trains were hauled by dinkey locomotives, mostly about

40-ton in size, although there were a few somewhat lighter, and three of them were 50-ton engines. Fifty-five to seventy pound rail was used in the track. It was found in most cases that the heavier rail was more satisfactory as it gave a better and more substantial track and required less maintenance, and resulted in less loss of time than the lighter weight rail. The difference in weight was not a serious objection even in case of track on waste banks which had to be moved often.

For distributing supplies and materials from the central warehouse, and for handling equipment repair jobs, most of which are "emergency," a fleet of motor trucks was required varying in size from the small box-body runabout or "light-delivery" car, to the 5-ton stake or dump body. During the peak of construction activities the truck fleet was made up about as follows:—

Light-delivery cars	19
1-ton trucks	12
2-ton trucks	7
5-ton trucks	6

Most of these trucks were kept at the central garage and sent out from there as needed,—one man gave all of his time to dispatching and keeping track of them, to keep them working to best advantage. Most jobs had one or more light-delivery cars assigned to them permanently, and occasionally a larger truck was so assigned either permanently or for a limited time.

Power Distribution Equipment

Electric power was brought to each job on high tension transmission lines. In some cases these were a part of the established distribution system of the power companies; in others, as at the dams, they were special lines built for the occasion. For each job the District had to furnish the step down transformers, lightning arresters, switches, etc., for the main transformer station, and the low voltage lines, transformers, etc. for distribution over the job. This required a large and varied amount of electrical equipment.

Miscellaneous Equipment

The usual equipment of derricks, hoists, pile drivers, unwatering pumps, air compressors, drills, etc., was required in addition to the equipment mentioned under specific headings. This mis-

cellaneous equipment requires no special mention, except as it will be spoken of hereafter in connection with specific operations.

Equipment Accounting

The equipment accounting was comparatively simple, and centered around an equipment card which is reproduced as Fig. 3. At the time a piece of equipment was received, one of these cards was filled out to represent it. On the heading, spaces are provided for designation (car, locomotive, motor, etc.), manufacturer, date of purchase, description, factory number, Conservancy number, and other information of minor importance.

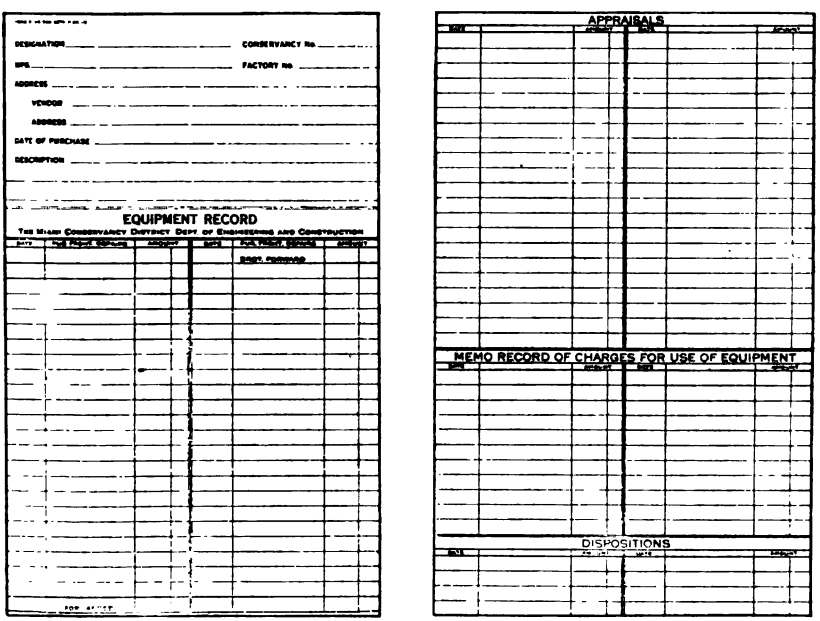


FIG. 3—EQUIPMENT CARD

One of these cards was made out to represent each piece of equipment on the work. The front and the back of the card is shown in the illustration.

There is space on the card also for a record of expenditures on the machine such as freight, repairs, hauling charges, etc. On the back of the card is a space for appraisals at various dates, as the machine depreciated or was transferred from one job to another. In fact this card provided for a complete record

of the machine from the time it appeared on the job, and one copy went with the machine wherever it was used. That is to say, if a locomotive was in use at the Englewood Dam, the equipment card representing that locomotive would be found in the files of the Division Engineer at Englewood. If the locomotive was transferred to the Hamilton work, the equipment card would then be taken out of the Englewood files and sent to Hamilton with the machine.

A duplicate copy of each equipment card was kept by the Accounting Division at the headquarters office, so that while each job had a record of its own equipment, the Accounting Division had the complete record of all the equipment belonging to the District.

When a piece of equipment was transferred to another job an appraisal of its value was made by the two division engineers interested, and the transfer was handled just as if it were a sale from one job to the other. The jobs involved in the transfer, having agreed to the transfer price, had no more comeback than if the equipment had been actually sold by one and bought by the other. Repairs or overhauling that might be necessary to put the equipment in good condition, would be agreed upon just as if the equipment was being purchased or sold outright. In case the two division engineers interested failed to agree on the transfer price, the matter was settled by the chief engineer's office, usually through the appointment of three disinterested employees as an appraisal board, whose report as approved or amended by the Chief Engineer would be final. After an overhauling, or extensive repairs, a new appraisal was usually made, so that the expense of heavy repairs could be spread out and not throw unit costs temporarily out of balance.

The original cost, plus freight, was used as a basis for depreciation as long as the equipment remained on the job to which it was originally assigned, or until a reappraisal was made. After transfer, or appraisal for other reasons, the new appraisal value was used as a basis for depreciation just as if the equipment had been sold by the transferring feature, or purchased by the receiving feature. When a piece of equipment was actually sold, the last feature having possession of it absorbed the difference between the selling price and the depreciated value. Towards the end of the job when the sale of equipment became an important matter, a Salvage Division was organized, and in

most cases equipment was transferred to the Salvage Division on the same basis as has been described for transfer between features, and the Salvage Division then absorbed the actual profit or loss as sales were made. This had the advantage of allowing the account for each feature to be closed, and final cost figures to be determined as soon as the work was finished and before all the equipment was actually sold, leaving the salvage account as the only one held open for final adjustment.

Cost of installation and dismantling was always charged to the feature for which this work was done. This cost was run through the plant account, to which depreciation of equipment was also charged, and was charged against construction items over a period of time so as not to have an unreasonable effect on unit costs from month to month. Here the distinction between *plant,* and *equipment,* should be made clear. Any expense for labor, materials, and supplies, and for *use* of equipment in the building of a plant to be used in the construction of some other structure or product, is a proper charge to *plant.* The term *equipment* applies to the non-expendable articles of machinery (often forming parts of the plant) which have a value separate and apart from the plant itself, and are capable of being transferred from plant to plant, or used independently. For example, a screening and washing outfit, as a working unit, capable of turning out washed sand and gravel, is a true plant. The motors, crushers, screens, etc. installed in the plant are items of equipment. A derrick set up and ready to work, is *plant.* The derrick irons and sticks, the hoist, and the motor or boiler, are items of *equipment.*

An arbitrary distinction was made between equipment and tools. On a large job it is practically impossible to keep track of small tools individually. It was the practice on this work to charge all such tools directly against the job for which they were purchased, just like other supplies. Whatever salvage value might remain in such tools, when transferred to another job, was credited to the job against which they were originally charged. In the classification adopted by the District, no article costing less than $100 was classed as equipment, and therefore no card was handled for such an article, even though it might meet the equipment classification in every other way. In the plant account, however, there was an item headed "miscellaneous tools and equipment" which took care of proper charges for use

of small tools and the smaller items of machinery or apparatus, which, because of their low value, were not given place in the equipment account.

The monthly charge for depreciation or use of equipment was fixed at 2% of the original cost including freight, or of the latest appraised value. In order to simplify equipment accounting this rate was applied to all classes of equipment regardless of the fact that it is probably too high for some classes, like electric motors, and perhaps a little low for apparatus like concrete mixers which usually get rough usage. The justification for the one standard rate was that it averaged about right for cost accounting purposes, and greatly simplified that part of the accounting work; also, however much in detail the depreciation rate might be worked out, it never could be exactly right, and there must always be an adjustment when equipment was finally sold. The difference in that final adjustment as between an elaborate system of depreciation and the simple one used, would have no appreciable effect on unit costs, or even on job costs.

The equipment charge began at the time the equipment was first put into use on the job, and continued until the equipment was transferred to another job, or was *listed* for sale or transfer. Thus the job was charged for idle time in case the equipment was being held by it for future work.

Minor repairs to equipment were charged direct to the item of work on which the equipment was working at the time; major repairs, however, were charged into the equipment account and added to the cost or taken up by a reappraisal, and then charged out, like first cost, on the 2% basis. Again for the purpose of simplifying accounting, the "major repairs" classification was limited to repair work costing $1000 or more, or amounting to 10% or more of the original cost of equipment whose value exceeded $500. All other repairs were charged direct to job items, just like supplies.

The total plant charge against a job therefore, was 2% per month for plant costs, plus the installation charge spread over the estimated time of use, plus an estimated charge for dismantling spread over a like period.

Details of plant accounting will be found in Part IX of the Technical Reports of the District, entitled "Accounting and Cost Keeping of the Department of Engineering and Construction."

CHAPTER II.—THE DRAGLINE EXCAVATOR

General

Of all the different types of excavating equipment in general use, the dragline excavator is now recognized as being best fitted to meet the requirements of many big jobs of excavation. Twenty years ago it was practically unknown. Today it is in general use in all parts of the United States wherever big construction work is in progress. It is so well adapted to so many different kinds of work, that it is coming into demand more and more, as its possibilities are being recognized. Its long reach enables it, in many cases, to deposit material direct from excavation into fill or waste bank, in one operation. Revolving on its base through a full circle, it can dig or dump anywhere within a radius determined by the length of its boom. It can build its own trackway ahead, and clean up behind as it goes. In many cases it can handle heavy loads as well as a derrick or locomotive crane. It can load cars almost, if not quite, as well as a steam shovel. It has been used successfully in rock cuts. It can pull down a bank standing high above the level at which it is working, and can excavate a pit far below where it stands. It can dig under water almost as well as in the dry. It can strip a gravel pit, placing the overburden off to one side, and then be ready to handle the gravel itself as required. To cross a river it simply builds a peninsula to serve as a track-way, then moves out on to it, takes up material from behind and deposits it ahead until the peninsula becomes a moving island, on which the machine travels on across until the other shore is reached.

The construction of the flood control project of the Miami Conservancy involved the excavation of 8,000,000 cu. yds. of material in widening and deepening river channels, and in excavation for outlet structures at the five dams. It required the placing of 12,000,000 cu. yds. of embankment in levees and dams. On account of unsettled conditions at the time of the beginning of construction (in 1918), due to the fact that this country was in

the midst of the World War, it was not practicable to handle the work by contract, and the District decided to undertake the work with its own forces, and to purchase its own equipment for handling the various construction operations. The dragline excavator was decided upon at once as the most suitable type of excavating equipment for many of the important features of its work, and as plans developed it turned out that time and again the dragline became the selection for this or that job, after other kinds of equipment had been considered and rejected. As a matter of fact there have been only a few of the excavating jobs on the whole Conservancy work, where the dragline machine has not been used successfully, and it has been very useful also on many items of work other than excavation.

In all, the District had at work, during the whole construction period, 21 of these machines, while several others were used by sub-contractors on various small units of the work. These machines varied in size from the little Marion Class 21, carrying a 30-foot boom and a ¾-yd. bucket, to the big Bucyrus Class 175, equipped with either a 100-foot boom and 5-yd. bucket, or a 135-foot boom and a 3½-yd. bucket. Table II gives a list of these machines, giving length of boom, size of bucket, type of mounting and other pertinent data.

Development of the Modern Machine

The Dragline Derrick. Prior to the time when the modern dragline excavator came into general use, many construction men had made use of movable stiff-leg derricks equipped with drag buckets, for handling certain jobs where conditions happened to be favorable for that type of equipment. Even today it is not uncommon to see such outfits performing certain work very economically. They have been used on the Conservancy work, in several places, with very satisfactory results. Fig. 4 shows an installation of that sort. A section of low embankment, 1½ miles long, for a portion of the relocation of the Ohio Electric Railway around the Huffman retarding basin, was to be built from side borrow on either side. No dragline machine was available for the work at the time it was desired to do it. The two stiff-leg derricks shown in this view were equipped with drag buckets, mounted on skids and rollers, and handled this job practically as well as a dragline machine could have done it. They backed out across the flats, side by side, building the fill as

Table II.—Miami Conservancy Dragline Excavators

Number of Machines In Use	Manufacturer	Class or Model	Power	Length of Boom Feet	Size of Bucket Cu. Yds.	Mounting
2	Bucyrus	175	Electric	100 to 135	5 to 3½	Truck
1	Lidgerwood	M	Electric	100	3½	Truck
4	Bucyrus	24	Electric	85 to 100	5 to 3½	Skids and Rollers
1	Bucyrus	24	Steam	85 to 100	5 to 3½	Skids and Rollers
1	Lidgerwood	K	Electric	100	3	Skids and Rollers
1	Lidgerwood	K	Electric	100	3	Truck
1	Bucyrus	14	Steam	60 to 75	2	Skids and Rollers
2	Bucyrus	14	Steam	60 to 75	2	Caterpillar
3	Lidgerwood	B	Steam	50 to 70	2 to 1½	Truck
3	Marion	36	Steam	40 to 50	1½	Caterpillar
1	Bucyrus	9½	Steam	45 to 60	1½ to 1¼	Caterpillar
1	Marion	21	Steam	30	¾	Caterpillar

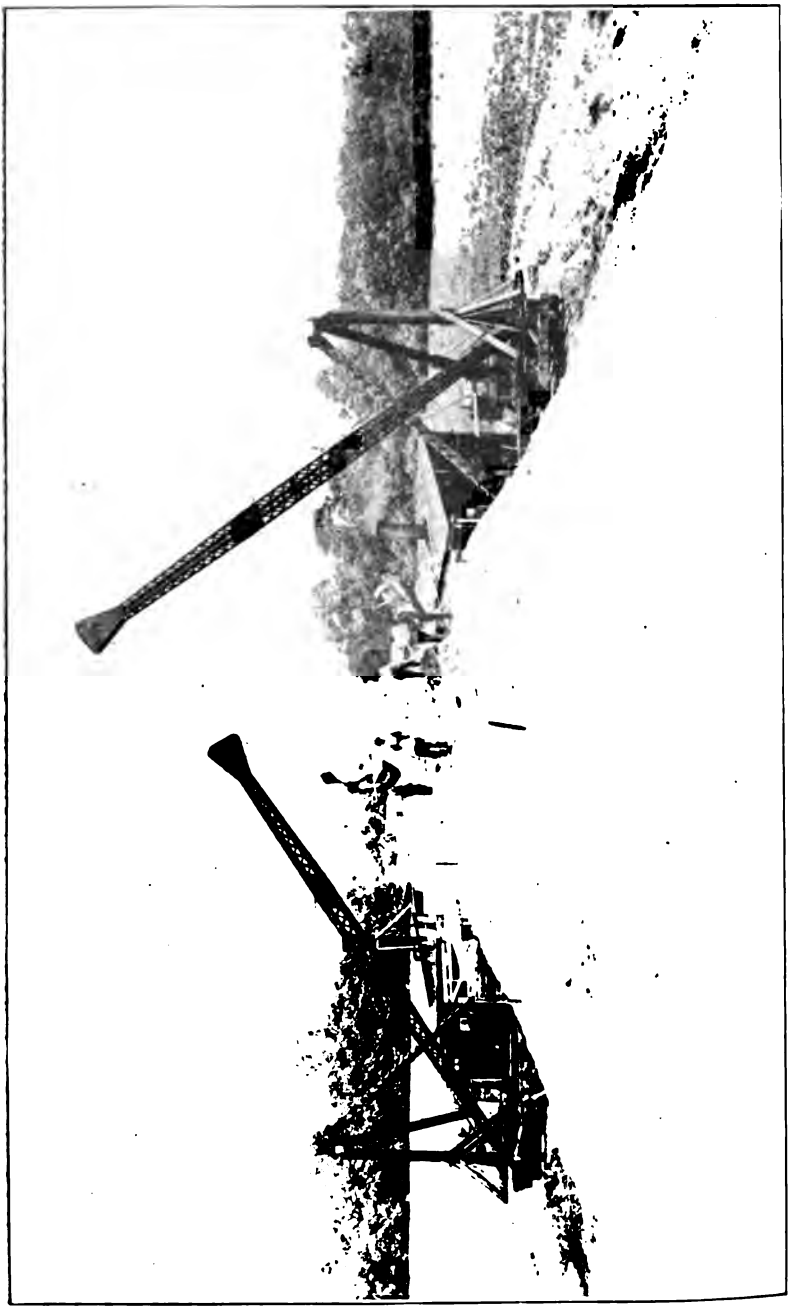


FIG. 4—DRAGLINE DERRICKS BUILDING EMBANKMENT

This embankment, a mile and a half long and containing 65,000 cubic yards of material, was built by these machines from side borrow at a cost of 27c per cubic yard, including the cost of equipping the derricks for drag-

they went. Much of the material in the borrow pits was taken from below ground water level, which in this case was close to the ground surface. The total quantity in this fill was 65,000 cu. yds., which was placed at an average cost of 27c, including equipment depreciation and all field overhead, with labor at 40c per hr. and other wages in proportion.

The swing of the boom in this case is only about 250°, which is a serious limitation for all-around work, compared with the up-to-date dragline, but it illustrates very well the first step in the development of the modern dragline excavator, and many jobs may still be handled efficiently with this type of equipment.

Fig. 5 shows one of the same machines excavating gravel for concrete, and is another illustration of an economical use of these improvised dragline derricks. The hopper used for loading wagons is also on skids, and can be moved ahead by the machine as its borrow pit is extended. Stripping of the pit, as required, is handled by the same machine in advance of the main excavation, as the machine is moved back, strip by strip, the overburden being dumped off to one side out of the way, or loaded into wagons and hauled to wherever it may be used economically.

The Timber Machine. The next step in the development of dragline excavators, was a machine similar to the modern dragline, except that it was built of timber, framed and assembled on the job. Except for the center casting, which had to be cast especially for the purpose, these machines were assembled from standard equipment parts, timber from the nearest source of supply, turn table and dollies from one place, boiler and hoist from another, swinging engines, derrick castings, sheaves and lines, all fitted up to the requirements as well as possible from standard parts originally built for other purposes. These machines, at least those of them which were intelligently assembled, were very successful, so much so, that the manufacture of standard dragline machines was soon undertaken, and naturally, a more finished and more durable machine was produced.

The Modern Machine. As compared with the equipment first described, the modern steel frame dragline excavator shown in Fig. 6 is a long step in advance. This is a Bucyrus, Class 24 electric, equipped with an 85-foot boom, and a 4½-yd. bucket, and illustrates the highest development in the dragline machine at the present time. The boom does not swing, as in the case of the dragline derrick, but is fixed with reference to the body of

the machine, thus permitting the hoist and drag cables to come straight in to the drums, and doing away with the constantly changing horizontal angles caused by a swinging boom. But the whole machine may be revolved through a complete circle, on the turn table just below the floor of the house. Thus its load, picked up at one point, may be swung around and dumped at any other point within reach of the boom. This machine is mounted

FIG. 5—DRAGLINE DERRICK LOADING GRAVEL

The gravel from the borrow pit is dumped into the loading hopper, through which it is fed by a hand-operated gate into dump wagons. The hopper itself is on skids and rollers and is moved ahead by the machine as the borrow pit is extended. Stripping of the pit as required is handled by the same machine in advance of the main excavation.

on skids and rollers; truck mounting may be had if preferred. Sectional mats made up of heavy timber are laid ahead on a prepared grade, to give firm and even bearing for the rollers. The grade is prepared by the machine itself, with a minimum amount of hand work by the two track men who are a part of the regular crew. As the machine moves ahead the mat sections left in the rear are picked up by the machine by means of eye bolts and

slings, and swung around into place ahead, where they are spotted by the track men. The bucket is then thrown out ahead, takes a bite into the ground, and the machine pulls itself ahead by taking in on the drag line, the rollers being manipulated by the track men, who also bring the rollers ahead from the rear as needed. When a truck mounting is furnished with these ma-

FIG. 6—MODERN DRAGLINE EXCAVATOR

This is a steel frame machine mounted on skids and rollers, equipped with an 85-ft. boom and a 4½-yd. bucket. The whole machine may be revolved through a complete circle on the turntable just below the floor of the house. The bucket is loaded by being pulled toward the machine by the horizontal cable leading in to the foot of the boom. The hoisting cable from the end of the boom lifts the bucket to the desired height, the bucket being so hung that it holds its load until the tension on the drag cable is released, when the nose of the bucket drops and the load is discharged. Thus a load may be picked up at any point within reach, swung around and dumped at any other point within reach of the boom.

chines a self propelling mechanism is usually a part of the regular equipment. In that case, sections of rail are mounted on the mats, and the mat sections must be set with more care, of course, in order to hold the gage.

Revolving Mechanism. The machine just described is revolved by means of a rack and pinion mechanism and can be turned in either direction any number of times, without limitation. There

is another type of revolving mechanism which is still in common use, but is gradually being displaced by the more efficient rack and pinion type. It is an adaptation of the bull-wheel, the swinging being accomplished by means of two cables having dead ends on the perimeter of the turn table, and leading around it in opposite directions. These cables run to drums, on the platform, which are operated by the swinging engine. While the machine

FIG. 7—SMALL DRAGLINE EXCAVATOR ON CATERPILLAR TRACTION

For the smaller machines this caterpillar mounting is desirable as it makes moving a quick and simple operation. The machine moves under its own power, and as it prepares its own roadway, moving can be done with practically no delay to digging operations. A machine thus equipped can also move quickly from one point to another some distance away.

thus equipped can revolve practically a full circle, it must return in the direction from which it has revolved. While ordinarily this is no great disadvantage, it is a serious limitation in some cases. The cable-swing machine is lighter and somewhat cheaper than the other.

Boom Suspension. The dragline boom may be raised or lowered to any position within working range by means of the boom line, which passes over a sheave at the top of the A-frame down

to a drum on the platform, which may be operated by power (on the larger machines), or by hand (on the smaller machines). It is not raised or lowered while working. For ordinary work it is set at an angle of about 30° with the horizontal. It requires only a few moments to change it to any position most advantageous to the work in hand.

Mountings. Table II, shows the three types of mounting of the Conservancy machines, and that indicates in a general way the mounting most suitable for machines of the various sizes. For the smaller machines, caterpillar mounting is most desirable, as it gives great flexibility and makes moving a quick and simple operation. It also does away with the necessity of track men to help handle and spot the mats. As the machine moves under its own power it simply has to level off a roadway for itself, and can move with almost no delay to digging operations. It can also move very quickly from one point to another on the job and may thus be available for other service than ordinary digging, on short notice. Fig. 7 shows a Marion Model 21 machine on caterpillar mounting, as used on the work at Hamilton. It was not uncommon for that machine to throw up a piece of track embankment, unload a car of coal, move a piece of track on the spoil bank, and load a car of scrap, with so little loss of time moving from job to job that all of these operations, or other similar ones, might easily be performed in one shift.

But the caterpillar mounting has its limitations, and it has not yet been worked out successfully for the larger machines. When we come to the size represented by the Class 24 Bucyrus, either skid-and-roller, or truck mounting is preferable. The former is the more common, and is very satisfactory for machines of medium size. These machines are not required to move as often as the smaller ones. They are usually installed for the purpose of carrying on a job of considerable size without interruption, and are not expected to move very far from job to job without being dismantled. The skid-and-roller mounting lends itself very well to meet most requirements of machines of that size.

The larger the machine the more rollers are required to support it. For the large machines, therefore, the number and weight of rollers required gets to be a handicap, and truck mounting is more satisfactory. The self-propelling feature, in many cases, is a marked advantage. That, of course, is impossible with skid-and-roller mounting.

The "walking" dragline as developed by the Monighan Machine Company, is so named because of its novel method of locomotion. When working, the machine stands on a large circular base. When ready to move it swings around to the proper position, then throws into gear a heavy steel shaft which passes through the machine from side to side. On each end of the shaft is a mammoth wheel segment which, when revolved, acts as a cam to lift the whole machine off the ground and forward about 8 feet. Each wheel segment carries with it, by means of carrying beam and chains, its own mat, or shoe, which drops into place on the ground, automatically, as the wheel segment comes down. And so the machine walks forward step by step, with its weight carried first on the center base then on the side shoes, very much like a man on crutches. Obviously, by the simple expedient of swinging around to the proper position when resting on its center base, this machine can start off in any direction, or can turn corners in one operation, since the wheel segment and side shoes swing with the body of the machine. This is an advantage of the walking dragline over other types. This type is not made in sizes larger than about 70-foot boom with 3-yd. to 3½-yd. bucket. The District had none of these walking machines of its own, but two of its sub-contractors used them successfully.

Power. The power for operating the dragline machine may be either steam, electric, oil or gasoline. Steam or electric is standard for machines of all sizes. Gasoline and oil engines are now used extensively on the smaller machines. The selection of power equipment necessarily depends on local conditions, such as transportation, hauling, and loading onto machine, for coal, and availability and character of supply for electric power.

On the Conservancy work an ample and dependable supply of electric power was available at reasonable rates, and electric power was preferred for most machines except the small ones, but the equipment was assembled under most unfavorable conditions because of the War and general confusion in the equipment market. It was necessary therefore to take what could be had within reasonable time, and as a result some steam machines were obtained which otherwise would have been electric. As a result of this, however, there was an excellent opportunity to compare steam and electric machines of the same type, working side by side on the same work. The story of that comparison is given in the latter part of this chapter.

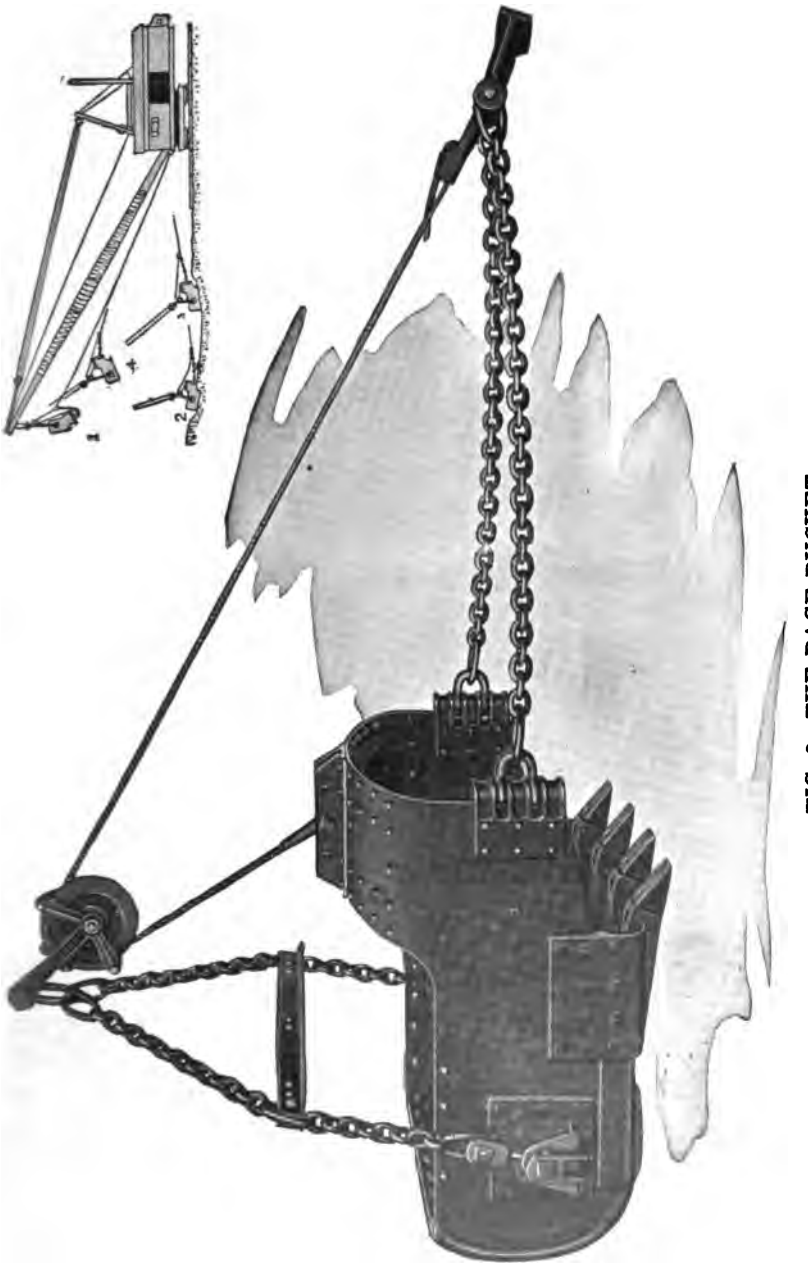


FIG. 8—THE PAGE BUCKET

This is the most common type of bucket used with dragline machines. The small diagram in the upper right hand corner shows the cycle of operations.

Bucket. Various types of scraper buckets have been built for dragline machines, but the one in most common use is the Page bucket, illustrated in Fig. 8. All of the buckets used on the Conservancy work were of this type, some of them slightly modified to fit the particular work at hand. For operating this bucket both the hoist line and the drag or load line are used. The bucket is so hung that when on the ground and being pulled in toward the machine by the drag line, it digs in and loads itself. Then the hoist line is put into play and lifts it out of the ground. By means of the short piece of cable running from the bail of the bucket over the sheave, at the hoist connection, and out to the drag connection, the bucket is held in a horizontal position as long as the drag line is kept taut. When the bucket is hoisted and swung to the desired position the drag line is released and the bucket dumps. These buckets are equipped with manganese steel lips, and except for soft material, manganese steel teeth are also used. These teeth are replaceable and need to be renewed frequently in hard digging.

The bucket shown in the illustration has a flat bottom. It has been found that in rock excavation a rounded bottom is preferable, as the bucket is less likely to turn over in loading. Some operators prefer the rounded bottom for general use. It cleans itself more easily in sticky material, may be made somewhat lighter for the same strength, and is better for trimming slopes and work of that sort. It loads in less distance than a flat bottom bucket. In fact, in many ways it has similar advantages to those of a round point over a square point shovel.

Fig. 9 gives an idea of the size of a 4½-yd. scraper bucket.

General Features of Operation

Crew. For ordinary work with a skid-and-roller machine a dragline crew consists of an operator, a fireman or oiler (on steam or electric machine respectively), and about 3 ground men who level up ahead for mats, spot the rollers for moving, look after water lines, load on coal, or look after electric cables, moving transformers, etc. The smaller machines on caterpillar mounting get along with only one or two ground men and sometimes none at all. On some of the very small machines the operator does his own firing when working conditions permit. The matter of adapting the size of the crew to the work at hand is one that needs some study on most jobs. It often happens

that under favorable conditions considerable economy can be effected by putting on an extra man or two, and thus increasing the output of the machine.

Power Consumption. Coal consumption per shift varies from 1 or 1½ tons for the small machines, to 4 or 5 tons for the large

FIG. 9—A 4½-YARD PAGE BUCKET

This illustrates the size of one of these large excavator buckets. Each of the men in the photograph is more than 6 feet tall. The bucket is standing on its nose and is held erect by the hoisting cable.

ones. The consumption of electric current will average somewhere near 1 K.W.-hour per cu. yd. for good material under fairly favorable conditions.

A comparison was made between two Class 24 Bucyrus machines, one steam and one electric, working side by side on the same work, over a four months period. During that time one machine excavated and loaded 283,000 cu. yds. of gravel, and the other 277,000 cu. yds. The steam machine averaged 4.1 lbs. of coal per cu. yd. and the electric 0.86 K.W.-hour per cu. yd. These were large machines, working under fairly favorable conditions.

A Class 14 Bucyrus steam machine, on 30,000 cu. yds. of good digging, averaged 7½ lbs. of coal per cu. yd. Obviously coal consumption per cu. yd. will vary more than that of electric power since fires must be kept up whether the machine is digging or not, while electric power is used only when the machine is actually working. Moreover the condition of boilers and engines is more likely to be allowed to run down than is that of corresponding parts of the electric machine.

A Bucyrus Class 175 electric machine in one month on channel excavation moved 52,000 cu. yds. with a power consumption of 1.05 K.W.-hours per cu. yd. The same machine during another month on better digging took out 88,500 cu. yds. with a power consumption of 0.92 K.W.-hour per cu. yd. This machine averaged 0.90 K.W.-hour per cu. yd. for a period of 16 months, during which time 980,000 cu. yds. of material was moved. Another Bucyrus Class 175 machine under less favorable conditions used 1.40 K.W.-hours per cu. yd. during one month moving 25,000 cu. yds., and 1.17 K.W.-hours per cu. yd. during another month when 69,000 cu. yds. were moved.

A Class 24 Bucyrus electric machine on rather favorable channel excavation work, loading into cars, used 0.75 K.W.-hour per cu. yd. during one month when 73,000 cu. yds. of material were moved. This same machine averaged 1 K.W.-hour per cu. yd. over a long period covering the handling of 2,500,000 cu. yds. Another Bucyrus Class 24 on borrow pit excavation at one of the dams loading into cars averaged 0.65 K.W.-hour over a long period.

A Class K Lidgerwood electric machine taking out 600,000 cu. yds. of borrow pit excavation at one of the dams averaged 0.65 K.W.-hour per cu. yd. Another Class K. Lidgerwood of the same type taking out 300,000 cu. yds. of channel excavation averaged 0.80 K.W.-hour. This same machine on 61,000 cu. yds. of more favorable work, averaged 0.61 K.W.-hour per cu. yd.

Cable Expense. Cable expense is a larger item of operation cost than is often supposed. Drag cables on the larger machines are good for not more than about 40,000 to 50,000 cu. yds. on an average, and on bad work or with careless handling may easily get down to 5,000. A fair service for hoist cables is 60,000 to 70,000 cu. yds. This item of expense will vary greatly from month to month, especially that for drag cables, as their life depends upon so many varying conditions and circumstances. A further discussion of cable use is given under an appropriate heading farther on in this chapter.

Supplies and Repairs. Table III gives a summarized list of supplies and repair parts, and cost of shop work on a Class 24 Bucyrus electric dragline machine during a year's operation on river channel excavation at Hamilton. This is of interest for two reasons: First, it shows the cost of certain items of supplies which are required in somewhat regular quantities, such as rollers, electric supplies, wire rope, grease, oil, waste, etc. Second, it calls attention to many replacements of parts which should be anticipated for a steady period of operation, not that all of these parts will need to be replaced each year,—the 4/0 flexible electric cable, for instance, was replaced only once during five years of work,—but in place of those that carry over for a longer period, other parts not mentioned here will be needed, so that this list may be taken as a suggestion of what may be required, and it represents an important item of operating expense.

During this period the machine handled 513,000 cu. yds. of excavation. This is considered a fairly representative year as far as output is concerned, and this particular list was selected because it also is considered fairly representative of the cost of upkeep of one of these machines in continuous service. It will be noted that a considerable portion of this expense is for expendable supplies, such as rollers, wire rope, repairs on bucket, etc., the cost of which will depend to some extent on the yardage moved during the period. The list includes all labor on repairs and upkeep outside of that furnished by the regular dragline crew.

Dismantling and Erection. Below are given some costs of dismantling and erection as they were obtained on the various jobs from time to time.

A Class 175 Bucyrus after being dismantled was hauled by motor truck through city streets, a distance of 3¼ miles, and

Table III.

Supplies, Repair Parts and Labor on Repairs and Upkeep for Class 24 Bucyrus Electric Dragline, During One Calendar Year.
(Excavation During This Period, 513,000 Cu. Yds.)

Supplies and Parts

Wood rollers— 94 @ $1.00			
259 @ .95			
198 @ 1.329			
196 @ 1.61		$ 919.66	
Brake Lining		695.12	
4/0 Flexible Wire Cable (Electric)		549.70	
Flexible Copper Wire		55.27	
Electrical Supplies		622.19	
Wire Rope Drag Lines	$1,533.99		
Hoist Lines	834.52		
Miscellaneous	11.55	2,400.06	
Grease, Oil and Waste		144.07	
Tools		58.62	
Fire Extinguishers and Lanterns		18.67	
Motor Bearing		13.50	
Intermediate Swing Shaft		35.00	
Vertical Swing Shaft Rack Pinion		175.00	
Hoist Friction Blocks		23.25	
Hoist Drum, Shaft and Gear		250.00	
Hoist Toggle Crank and Lever		10.50	
Hoist Friction Band with Blocks		150.00	
Jaw Lever, etc., Hoist Drum		31.10	
200 H.P. Stator for Motor		1,200.00	
Drag Drum and Gear		600.00	
1½-in. Drag Chain (for bucket)		96.96	
Gear Segment		100.00	
Heating Stove		18.52	
Cradle Base		30.00	
Brake Valve		19.65	
Cross Head		23.00	
Adjustable Bolts		82.14	
Bronze Bushings		125.51	
Hoist Drum Shaft		197.00	
1 Swing Shaft		28.91	
3 Intermediate Shafts		117.43	
6 Large Shackles		92.41	
176 1⅛-in. Machine Bolts		123.65	
Miscellaneous Bolts, Clamps, Steel, etc		929.63	$9,936.52

Shop Work

(Labor by Regular Dragline Crew Not Included)

Repair Hoist Rotor on 200-H.P. 440-Volt Motor	$ 10.84
" " " " " "	43.90

Table III—(Continued)

Repair 1 Fan for Slip Ring End of 200-H.P. Rotor....	8.23	
Rebabbitt and Bore 1 5-in. Motor Bearing and 1 3½-in. Motor Bearing	38.95	
Repair Hoist Rotor	14.65	
Repair Rotor for 200-H.P. Motor	22.57	
Repair Box for Shipping Rotor	8.33	
Repair Phase Straps on Hoist Rotor, 200-H.P.	100.50	
Repair 4½-yd. Bucket	829.02	
Repair 200-H.P. Rotor	49.44	
" " "	10.93	
" " "	29.73	
" " "	18.33	
Crane Service Handling 200-H.P. Rotor	3.87	
Repair Air Compressor	7.47	
Repair Bronze Slip Ring	5.04	
Repair 200-H.P. Westinghouse Stator	147.69	
Load 200-H.P. Rotor on Auto Truck	6.91	
Repair 200-H.P. Rotor	125.75	
Take Pinion Off Rotor	17.84	
Electrical Work	79.52	
Install 5-H.P. Motor on Air Compressor	72.54	
Repair Swing Shaft	14.99	$ 1,667.04
		$11,603.56

then re-erected. The cost of dismantling was $2,250, cost of hauling $687, and the cost of re-erection $4,880.

A Class K Lidgerwood machine was dismantled, hauled by truck over city streets, a distance of about 2½ miles, and set up again for further work. In this case the cost of hauling was $350, and the dismantling and installation cost $3,316.

A Class K Lidgerwood machine was dismantled, hauled about ½ mile by truck and loaded on cars at a cost of about $3,250. To handle this machine it was necessary to erect two derricks, one at the machine for dismantling, and one at the railroad siding for loading.

A Class 24 Bucyrus machine was unloaded from cars and erected, by means of a derrick set up for the purpose, at a cost of about $2,500.

Another Class 24 Bucyrus was unloaded and erected at a cost of about $3,300. The work was done by hand with the aid of gin poles.

A Class 24 Bucyrus was dismantled and loaded onto cars at a cost of $1,250. In this case the machine stood alongside a rail-

road siding and a small Model 21 Marion was used for dismantling and loading.

Another Class 24 Bucyrus was dismantled and loaded, with the help of a guy derrick, for about $2,250.

Another Class 24 Bucyrus was dismantled and loaded at a cost of about $1,950, of which $100 was for freight on a locomotive crane.

A Class B Lidgerwood machine was dismantled and loaded with the help of a locomotive crane at a cost of $430.

A Class 14 Bucyrus machine on caterpillars was unloaded from car and erected at a cost of about $1,600.

Another Class 14 Bucyrus, on skids and rollers, was dismantled, hauled about 22 miles over good roads and erected, at a total cost of $2,447, of which $744 was for dismantling and loading on trucks, $523 for hauling, and $1,210 for re-erection. In this case a Model 21 Marion was used for re-erection, for which a charge of $160 is included.

Output. The average monthly output of a Class 175 Bucyrus machine (100-foot boom, 5-yd. bucket) ought to be from 70,000 to 80,000 cu. yds., working two 10-hour shifts on fairly good digging. The maximum may be greatly in excess of this, but the average takes into account shutdowns for repairs, overhauling, moving and other interruptions. The Class 24 Bucyrus (85-foot boom, 4½-yd bucket) will average 60,000 to 70,000 cu. yds. per month, under similar conditions. The Class B Lidgerwood (60-foot boom, 2-yd. bucket) is good for a monthly average of about 20,000 to 25,000 cu. yds. The smaller machines on the District work, like the Bucyrus 9½, or the Marion 21, were not often held on an excavating job for any great length of time, there were so many special jobs for them to do. The Marion 21, (30-foot boom, ¾-yd. bucket) will handle about 30 to 40 cu. yds. per hour on straightaway work.

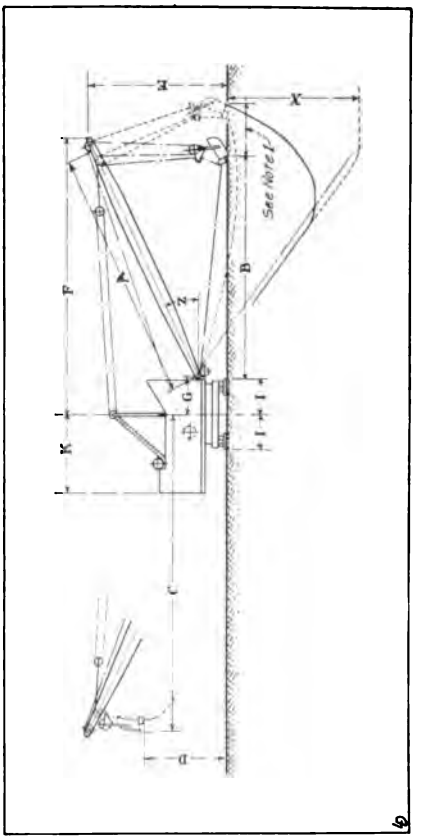
The foregoing gives a general idea of what may be expected as an average output from the machines of various sizes, working in good material. Daily and monthly records much larger than these are often secured, and for short jobs the machines may be good for nearly twice as much as the figures given would indicate. There is a wide difference, however, between maximum and average performance on work of this sort, and it has been attempted here to give only a general idea of what may be expected in the long run, under the conditions stated.

Table IV, giving working dimensions of Bucyrus machines of various sizes, is taken from a recent catalog of the Bucyrus Company.

Performance

Standard Work. Elsewhere in this report there will be found reference to the performance of the dragline machines on the various kinds of excavation work which were scheduled items of the District's construction program. The excavation for the outlet works involved the removal of both earth and rock in large quantities. Some of this was a cut and waste proposition, some was loaded into cars and hauled away. The borrow pits for the dams with one exception, were in large deposits of river gravel with an overburden of loam and silt. Much of this borrow pit excavation was below water level. All of it that was handled by draglines, was loaded into cars and hauled to the dams. The channel excavation was wet work largely. Some of it was placed directly into levees, or waste banks. In many cases this was accomplished by moving the material two or three times from excavation to levee or bank. Some of it was loaded into cars and hauled away. Many miles of levee were built by draglines from material taken from borrow pits alongside. All of this is standard dragline work, and no further description of it is needed here as it will be discussed in detail under appropriate headings in another chapter. There were, however, in addition to these standard excavation jobs, a great many special jobs performed by the dragline machines, some of which are worthy of special mention as illustrating the various uses to which the machine is adapted.

Dragline Dredge. One section of the channel work through Dayton was of such character that ordinary dragline operations were not entirely practicable. There was no chance to place the excavated material into levees or waste banks even by moving several times. It had to be hauled away. A car and track proposition would have been very expensive and difficult to manage, and would have been subject to frequent damage from high water, because of the river channel conditions along that section. Most of the material in question was below water level, and practically all of the work was shallow cutting. It was finally decided to mount a dragline machine on a spud-scow, load the excavated material on to other scows, to be towed to available spoil banks and unloaded by another dragline standing on shore. Con-



OUTLINE SKETCH OF BUCYRUS DRAGLINE EXCAVATORS
(See Table IV on next page).

TABLE IV

Working Dimensions of Bucyrus Dragline Excavators

Boom—	CLASS 7		CLASS 9½		CLASS 14		CLASS 20		CLASS 24		CLASS 175	
A....	42 feet		45 feet		60 feet		85 feet		100 feet		125 feet	
Angle—												
Z.....	25°	40°	25°	40°	25°	40°	25°	40°	25°	40°	25°	40°
B.....	40′– 0″	34′– 0″	43′–6″	37′– 6″	61′–8″	53′–0″	87′–9″	75′– 9″	104′–0″	90′– 0″	125′–3″	109′– 0″
C.....	42′– 0″	36′– 0″	48′–6″	40′– 6″	65′–9″	58′–0″	92′–2″	80′– 0″	108′–6″	94′– 7″	129′–9″	113′– 8″
D.....	10′– 6″	19′– 8″	13′–6″	23′– 6″	15′–6″	28′–0″	26′–6″	45′– 4″	28′–8″	50′–10″	40′–4″	67′– 4″
E.....	23′– 0″	32′– 7″	27′–9″	38′– 3″	34′–0″	47′–9″	45′–9″	65′– 1″	51′–3″	74′– 0″	62′–0″	90′– 4″
F.....	43′– 6″	37′– 6″	49′–6″	42′–11″	67′–2″	58′–6″	92′–0″	79′–11″	108′–4″	94′– 5″	130′–2″	113′– 0″
G.....	4′– 2½″	4′– 2½″	7′–2″	7′– 2″	8′–8″	8′–8″	11′–7″	11′– 7″	13′–2″	13′– 2″	12′–10″	12′–10″
I.....	6′–10″	6′–10″	6′–9″	6′– 9″	8′–5″	8′–5″	11′–9″	11′– 9″	13′–5″	13′– 5″	13′–10″	13′–10″
K.....	14′–10⅝″	14′–10⅝″	17′–8″	17′– 8″	19′–1″	19′–1″	23′–6″	23′– 6″	27′–2″	27′– 2″	27′–2″	27′– 2″
(Note 2)												
X....	14′–18′		15′–18′		20′–25′		30′–35′		35′–40′		45′–50′	
(Note 3)												
Y....	22′– 6″	14′	20′	13′	32′	20′	58′	40′	58′	36′	69′	43′

Note 1—A good operator can throw the bucket from 10 to 40 feet beyond the end of the boom depending upon the size of the machine and the conditions under which it is working.

Note 2—The digging depth "X" does not conform to any fixed rule. It increases with the length of boom, but varies greatly with the material being dug. In heavy clay the machines will dig as shown in the table. Under conditions where they can be held close to the edge of the bank and where the material fills the bucket easily these figures may be considerably larger.

Note 3—"Y" is the depth below level of machine to which the standard length of hoisting rope will allow the bucket to reach.

sidering the inherent difficulties of the job, this worked out very well.

Fig. 10 shows the "fleet," and the work under way. The dragline machine was a Class 175 Bucyrus electric, equipped with a 100-foot boom and a 5-cu. yd. bucket. The scows and the tug boat were built by the District at its temporary "shipyard" on

FIG. 10—DRAGLINE-DREDGE, SCOWS AND TUG-BOAT, EXCAVATING RIVER CHANNEL AT DAYTON

The scow at the left is being loaded by the floating dragline. The scow at the right is being swung into mooring position temporarily until the loading of the other one is completed. Another dragline machine on shore at the left end of the concrete bridge is unloading another scow and placing the material in the spoil bank. The tug-boat and scows were built on the job.

the river bank at Dayton. The spud-scow was 40 ft. x 80 ft. x 7 ft. with four 24-in. x 24-in. spuds, one at each corner. The dirt scows were 40 ft. x 120 ft. x 6 ft., decked over. Three of them were required. The tug boat was 20 ft. x 70 ft. x 3.5 ft., flat boat, stern-wheel type, operated by steam. In order to get floatation at low water, it was necessary to build a low temporary dam just below the lower end of this section of work.

In mounting the dragline on its scow the trucks and sub-base were dispensed with, so that the center of gravity of the outfit

was lowered as much as possible. A plan was worked out for reversing the bucket, so that it would load by being pulled away from the machine. It was thought that that would avoid piling up loose material just in front of the scow. As a matter of fact, the regular method of loading the bucket, that is, by pulling it toward the machine, gave no trouble, and the scheme for reversing the bucket was never put into use.

An ordinary load for the deck-scows was 200 to 250 cu. yds. The maximum length of haul to the spoil bank was about 6,000 feet with an average haul of about 3,000 feet. There were two bridges crossing the river along this section of the work which slowed up towing operations somewhat, especially during windy days and during periods of high water.

The loading of the scows was an ordinary dragline operation, slowed up of course by the fact that all excavation was shallow cutting, under water, and that the completed channel had to be reasonably close to the required lines and grades. The unloading, by machine on shore, was accomplished with a flat bottom bucket without teeth, taking off as much as possible of the load and throwing it directly into the spoil bank. Naturally some of the material was dragged off into the water during this process and that was picked up again and disposed of between trips. To finish the unloading a timber drag, hung from the bucket, was used to scrape off the scatterings. At first a low timber bulwark was built near the edge, along one side of the scow, to afford a solid face against which the bucket could work in unloading. It was found, however, that some material was always left along the sides of the bulwark, resulting in wasted space and causing the scow to list badly when empty. This added to the draft and made it harder to manipulate, so the bulwarks were removed. As a matter of fact the unloading machine always had time to clean up, because navigation troubles and other delays when they occurred, affected the other units much more than it. By working upstream with the spud-scow and letting the deck-scow trail downstream, it was easy to manipulate the latter for loading, with one line. The tug would bring the empty down within reach of the dragline machine, which would shift the scow into exact position. At the unloading point the scow was held fast by lines from the shore.

About 350,000 cu. yds. of material were moved by this floating equipment in the manner just described, and the deck-scows and

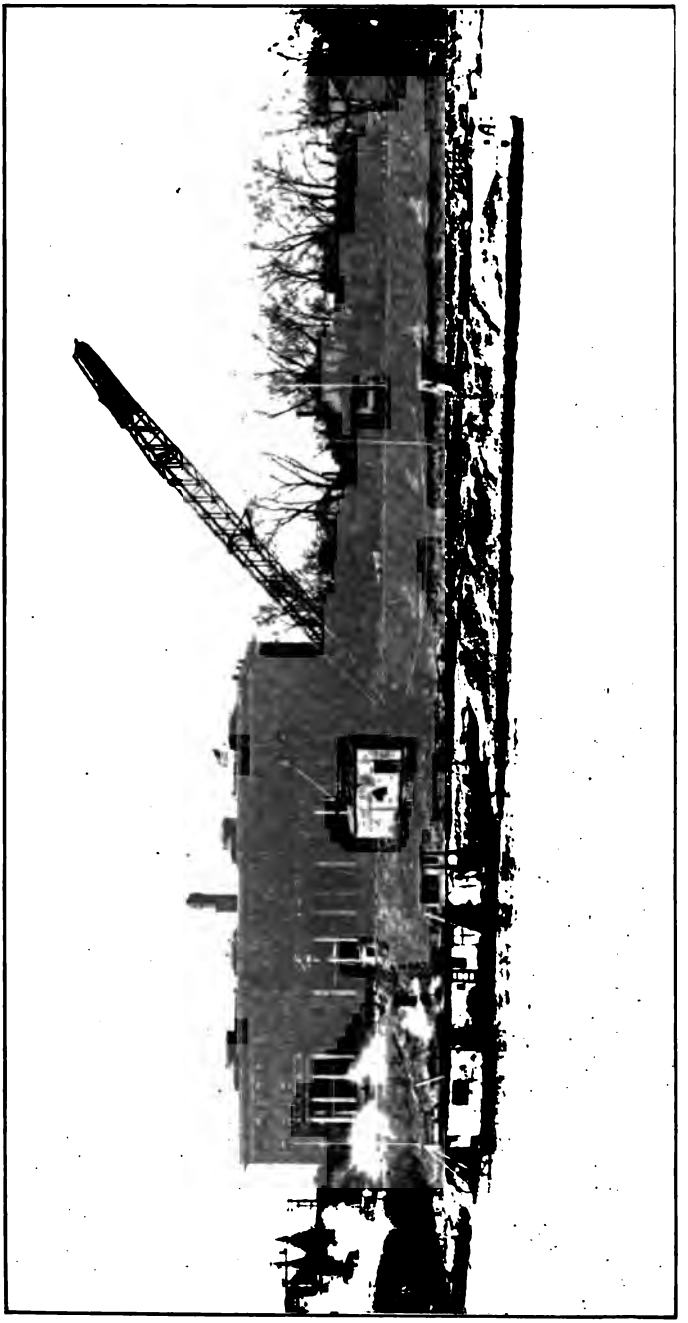


FIG. 11—LOADING SCOWS FROM THE SHORE

On some of the work a dragline machine working from the shore loaded the scows for transportation to the spoil banks, while the unloading was done by another machine as shown in the previous illustration.

tug boat were also used for transporting much other material which was loaded by a dragline working on the shore, as shown in Fig. 11. Field costs during a typical month show a unit cost for this work of 73c per cu. yd. sub-divided as follows:

Loading scows	20c	per	cu.	yd.
Transporting	26c	"	"	"
Unloading	19c	"	"	"
Distribution General Items	08c	"	"	"
Total field cost	73c	"	"	"

to which about 3c should be added for overhead and general expense. Plant charge, field engineering and inspection are included, but no allowance for extra expense due to winter work and extraordinary delays.

At the time this work was being done common labor was paid 40c to 46c per hour, dragline operators $250 to $280 per month and electric power cost about 1.9c per K.W.-hour put on the District's distribution lines. This was a piece of work that involved shallow cutting, much moving, long hauls, and was hard to do with any kind of equipment. The average output of the dragline on this work was 19,000 cu. yds. per month and the maximum, 55,400 cu. yds. per month, while on similar river channel work, but where the machine was on land and could cut and waste, the same machine averaged 40,200 cu. yds. per month and had a maximum monthly output of 69,100 cu. yds.

Another combination of dragline machine and scow is shown in Fig. 12. This is a Bucyrus Class 9½ machine on caterpillars, with a small spud-scow, 30 ft. x 69 ft. x 6 ft., on which it was mounted for odds and ends of river work, or was transported up and down river for work ashore. This outfit did a great variety of work such as helping to lower or change pipe lines crossing the river; cleaning out under bridges or around bridge piers at points where the big machines could not reach conveniently; excavating for structures at sewer outlets; building temporary dam to obtain slack water for floating equipment; pile driving; excavating gravel for concrete; and many miscellaneous jobs for cleaning up. It was very convenient largely because of its mobility, as it could work either from the water side or from the shore side as occasions demanded. It could dock almost anywhere along the river by digging a pilot channel for itself in to the shore.

Lowering Pipe Crossings. Across the river channel in Dayton were thirteen pipe lines—water, sewer and gas—which had to be lowered to make room for the improved channel. Having the dragline machines available, they were used to good advantage for these jobs and it is doubtful if the work could have been done as cheaply or as quickly with any other type of equipment.

Having located a pipe line, a trench was dug as close as possible on each side of it with a dragline bucket, and the material excavated was used for a cofferdam around the section to be lowered, the track fill upon which the dragline traveled being utilized as one side of the cofferdam. When it was necessary to lower the pipe the whole width of the river, the work was usually divided into two sections of approximately equal lengths, so as not to obstruct the flow of the river to too great an extent. The narrowest trench that could be excavated by the dragline bucket was considerably wider than was required for the actual pipe work, but the extra width thus made available was convenient for the necessary hand work in completing the excavation around the pipe or in handling the pipe itself. Unwatering pumps, pipes, etc., were easily moved from place to place by the dragline machine. Sumps were readily excavated to any required depth, and altogether the procedure was very satisfactory. More material was excavated than would have been the case if sheeted trenches had been dug or if open trenches had been dug by hand, but the advantage gained in doing away with the sheeting and having ready at all times a powerful and positive excavator, much more than offset the disadvantage of handling the extra material. Notwithstanding the large quantities, the unit cost was so much less than could have been achieved by any other means, that a considerable saving was made on the job as a whole.

Fig. 13 shows a large dragline machine lowering a 12-in. water main. After the trenches on either side were excavated, men with shovels carefully lowered the pipe by digging the material from beneath it. In this case the pipe was continued in service during the whole process of lowering, although the pressure was somewhat reduced temporarily, as a matter of precaution.

It was not always economical to tie up one of the larger machines on a job of this sort and in many cases the work was done in a similar manner with a smaller machine like that shown in Fig. 12.

FIG. 12—CATERPILLAR DRAGLINE MOUNTED ON SCOW

This combination was very useful for performing odds and ends of river work like cleaning out under bridges or around bridge piers, excavating for structures at sewer outlets, and other miscellaneous jobs where a larger machine was not available. The machine was thus easily transported up and down the river and could work either from the scow or from the shore as occasion demanded.

Building Bridge Piers. At Hamilton, the District's forces built for the city a 7-span reinforced concrete bridge over the Miami River at Black Street. A Bucyrus Class 14 dragline, mounted on caterpillars, and equipped with a 75-foot boom and a 1½-yd. bucket, was used to excavate for the piers and abutments, and to drive piling for the pier foundations. Notwithstanding the fact that the excavation for the piers had to be carried down 15 feet or more below the river bed, the work was accomplished successfully in open cut by the dragline machine, and a considerable saving in time and cost was effected. In one case it was possible to excavate for two piers within the same cofferdam.

The excavated material was thrown up around the excavation to form a cofferdam, the machine moving around on top of the pile. The unwatering equipment was also handled conveniently by the dragline machine. As soon as the excavation was completed, the foundation piling was driven. A steam hammer and its leads were suspended from the point of the boom, which was lowered to approximately a horizontal position. The hoist line was attached to the hammer for raising it in the leads, while the drag cable, passing through a pulley at the end of the boom, was used to snake over the piles and swing them into place in the leads. It was a very simple matter to spot the piles, and then by allowing the end of the boom to rest on top of the leads the bottom was forced a foot or two into the gravel, and the whole was held rigidly in place, eliminating the necessity for guy cables. Steam for the hammer was supplied by the boiler on the dragline. Fig. 14 shows this work under way. The foundation piles for the nearest pier are just being driven. The excavation has been made by the same dragline machine. The pier at the right is shown with the footing poured, and the forms are in place for the pier itself. Concrete for the footing was mixed on the site (the mixer may be seen just at the rear of the right-hand pier) and the concrete was also placed in the footing by the dragline machine. Later a cableway was erected, from which the remainder of the piers and the bridge itself were poured. After the concrete was placed in the piers and the forms removed, the dragline machine cleaned up the cofferdam and used the material for back-fill around the piers.

All these operations were handled effectively with the dragline, with a minimum risk of interruption or damage to equipment by floods. In case of high water which might overtop the coffer-

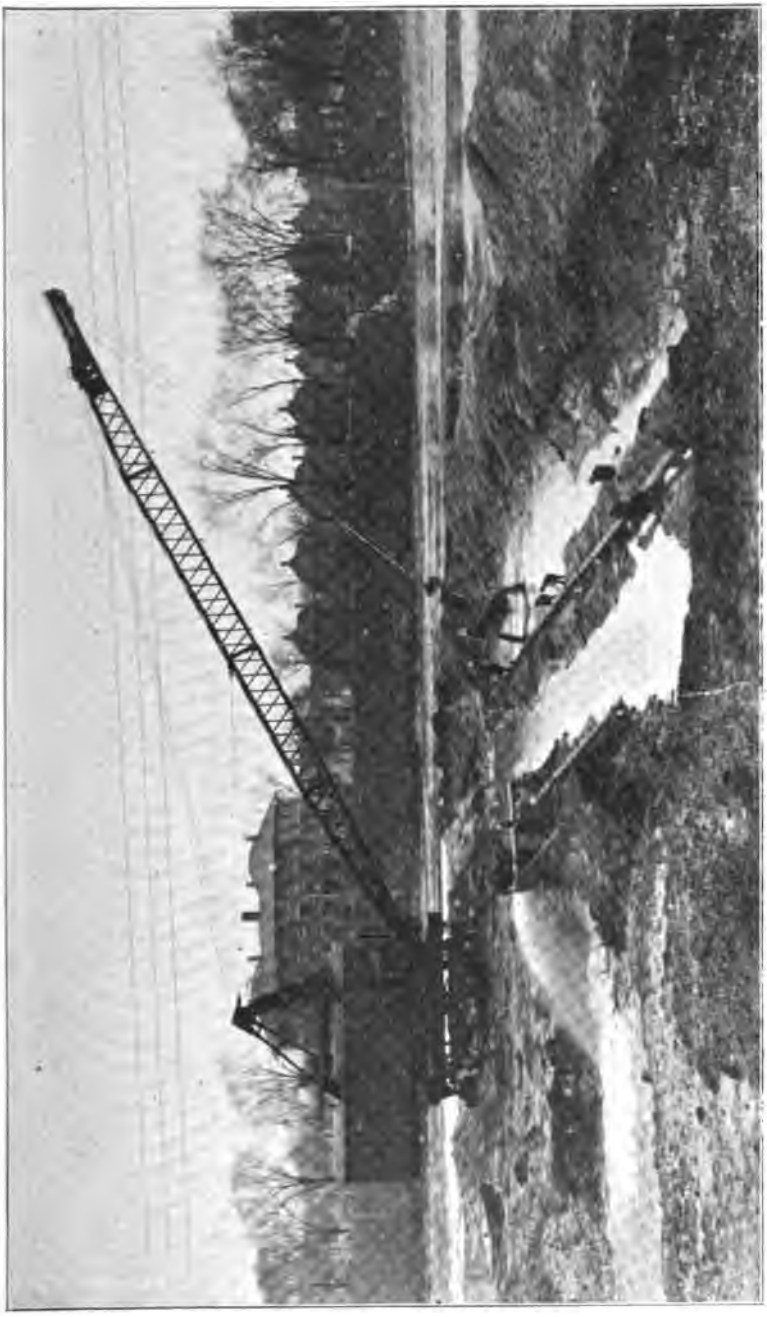


FIG. 13—LOWERING A WATER PIPE ACROSS THE MIAMI RIVER AT DAYTON

This pipe was maintained in operation during the whole time that it was being lowered to its new position. The dragline machine performed the rough excavation, and the actual lowering was done by hand in a minimum amount of time.

FIG. 14—EXCAVATING AND DRIVING PILES FOR BRIDGE PIERS

This dragline machine made the excavation in open cut for the piers of a large seven-span reinforced concrete highway bridge across the Miami River at Hamilton. The piles for the pier foundations were also driven by the same machine by hanging a steam hammer and set of leads from the end of the boom. In one 10-hour shift 52 piles were driven to 18 ft. penetration.

dam, the motors for the pump, or the whole pumping outfit, could be lifted out of place by the dragline machine, and then the machine itself could move over to shore on top of the cofferdam and remain ashore in safety until the flood had passed.

The unit cost of this excavation was figured on the basis of what would have been included within the usual sheet pile cofferdam. In other words, no allowance was made for the extra excavation required for side slopes. On that basis the unit cost of excavation was $2.11 per cu. yd. This is cheaper than it could

FIG. 15—DRAGLINE MACHINE BUILDING DAM

The first section of the dam at the far end has been completed, and the river diverted through the five gaps which were left for that purpose. The excavation for the remainder of the dam has been done by the dragline, building the cofferdam at the same time. The machine is now driving sheet piling. The round piles at the downstream toe have already been driven in a similar manner.

have been done within sheeting, the time and expense of sheeting was saved, and the time of completing the whole job was much lessened by the open cut method. As to pile driving, 52 piles were driven to 18 ft. penetration in one 10-hour shift. The unit cost including the piles themselves, was 58c per lineal foot. The piles were of native hardwood and cost 28c per foot delivered on the job. Unit costs include plant charge and field supervision.

At the time this work was done, common labor was receiving 46c per hour; carpenters 65c to 75c; dragline operators $280 per month.

Dam Building. Previous mention has been made of the construction of a low temporary dam at Dayton, to afford slack water for the floating equipment there. The small dragline machine and scow shown in Fig. 12 was used to advantage on the excavation, pile driving, and cofferdams on that work. Fig. 15 shows a dragline at Hamilton in use on the construction of the Two Mile dam there. This is a low concrete overflow dam five to nine feet high, built as a barrier to prevent the drift of gravel into the improved channel. The dam has an apron downstream of concrete and concrete blocks, and a short apron upstream. There is a row of steel sheet piling 8 to 12 feet deep along the edge of the upstream apron, and a row of round timber piles 15 feet long driven 3 feet center to center along the downstream edge of the concrete apron. The overall width of excavation (up and down-stream) is 72 feet.

The dragline started work on the far end of the dam and worked toward the center of the river, taking as the first section about half the width of the river. The excavated material was thrown up on either side to form cofferdams. An 8-in. unwatering pump was sufficient to control the seepage. After the excavation had been finished, except for what hand work was necessary, the dragline drove the row of timber piles in the same manner as had been described for similar work at the Black Street bridge. The steel sheet piling along the upper toe was then driven as illustrated in Fig 11. In this view the first section of the dam had been completed, the cofferdams removed (by dragline), and the flow of the river turned through the five gaps in that section of the dam left for that purpose.

The progress in driving the round piles was about the same as that made at the Black Street bridge. An average day's work for the driving of sheet piles was about 30 pieces per 10-hour shift, 8 to 12 feet penetration.

Plant Erection. In numerous cases a dragline machine has been used in place of a crane or derrick for the erection of larger machines or other construction plant. At one point three dragline machines were to be set up, one small one and two larger ones. The small one was erected with the help of gin poles, and was immediately put into service for erection of the two larger machines. This small machine was a Model 36 Marion on caterpillar traction, and was most useful in the erection of the two larger machines, saving greatly in both cost and time of erection.

At another place one of the large dragline machines was caught in a flood, undermined and sunk in the river bed to a depth of about 15 feet at one corner. A small machine on caterpillars was sent down to the wreck, threw a cofferdam around it and was used to excellent advantage in dismantling, righting and re-erecting the wrecked machine. The saving in time and expense in this case was very great over any other method that could have been used.

In numerous instances one of the small machines has been used to dismantle and load for shipment another dragline machine or other pieces of equipment.

Shifting Track. A Model 21 Marion dragline on caterpillar traction at Hamilton was used successfully for shifting track on the dump. This dump was an extensive spoil bank where hundreds of thousands of yards of material were wasted from channel excavation. The material was brought in on 12-yard dump cars and distributed by a spreader, so when track was shifted it was moved over about 12 feet. For moving track the dragline machine was pulled up close to the track, the line of travel of the caterpillar being parallel with the line of track. A set of tongs was attached to the bucket and the machine was turned so that the boom extended over the track in the direction in which it was to be shifted. The tongs were fastened to the rail and when a section of the track was lifted, it swung out naturally away from the machine, due to the position of the boom and the manner of operation as illustrated in Fig. 16. This first bite accomplished a move of from 2 to 3 feet, the dragline then pulled ahead and repeated the operation until the whole line had been moved over that distance. It was necessary to move back and make 3 or 4 runs to accomplish the full shift of approximately 12 feet.

The time required for this operation was really very short, and this machine was quite effective for that purpose.

Building Track Grades. On several parts of the work small dragline machines were used effectively for building grades for construction tracks. For this purpose machines with caterpillar traction were usually selected. Materials for fills were usually borrowed alongside, and in the few cuts that were required, it was usually possible to waste the material. Good progress was made on this kind of work, and low costs were secured. In one case a stretch of 2,900 lineal feet of embankment for a standard gage construction track, containing 3,000 cubic yards, was built

by this method, at a labor cost of about $1,000. In another case, a section of track embankment containing 40,000 cubic yards was put in at a labor cost of $5,130. This was accomplished in 55 working days.

Building Roadways. It hardly seems necessary to call attention to the advantage of small dragline machines in building roadways, runways and approaches. There was much of this kind of work to be done in connection with the Miami Conserv-

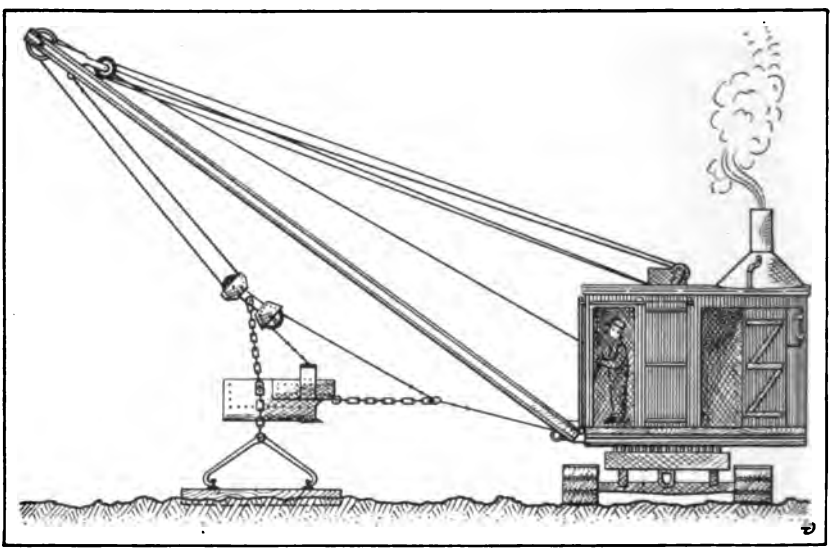


FIG. 16—DRAGLINE MACHINE SHIFTING TRACK

By this operation the track was shifted a distance of 2 or 3 feet, section by section. Then the machine moved back and made 3 or 4 additional runs in order to accomplish the full shift of approximately 12 feet. This method of shifting track turned out to be both speedy and economical.

ancy project, and the small dragline machines were continually being called upon for such service. The three Model 36 Marion machines were all convertible into steam shovels, and one shovel boom and dipper was purchased with this equipment and used whenever either of the three machines required it. In some places in making roadway cuts and runways, the shovel boom was more desirable than the dragline equipment, but in most instances the machines were used as draglines.

Unloading Cars. For unloading cars where a derrick is not available, a small dragline machine may often be very useful. At Hamilton, a Model 21 Marion was equipped with a clam shell bucket, and used for unloading coal. This particular operation, however, was not an outstanding success and probably was not much cheaper than unloading by hand, although it was considerably faster.

It is evident, however, that for loading or unloading pipe, heavy machinery, etc., a dragline machine, in lieu of a derrick, is a most useful piece of apparatus.

Loading Wagons. A Model 21 Marion at Hamilton was also used for loading top soil into wagons for distribution on levee slopes. For this purpose the regular ¾-yard drag bucket was used, two loads being required to load a wagon. With a first-class operator there was very little difficulty in spotting the bucket for dumping, and the operation worked successfully from the start. An average of 20 wagons per hour was a good average day's work. The only difficulty was to supply enough wagons to keep the dragline machine busy.

Back-haul Under Bridges. The construction program at Dayton was so arranged that both Class 175 Bucyrus machines were at Herman Avenue bridge at the same time, one on the upstream and the other on the downstream side. This was taken advantage of, in excavating underneath a portion of the bridge, a "back-haul" scheme being adopted. The back sides of the two buckets were tied together with about twenty feet of cable, so that when one machine pulled its bucket toward itself, the other bucket was at the same time drawn away from its machine. Thus, while one bucket was being filled and bringing its load out on one side of the bridge, the other was being drawn back into position for filling. The operation was entirely successful.

Dragline Cable

General Discussion. Wire rope is used extensively on dragline machines, principally for boom fall, for hoist line, and for drag line. The two latter need to be replaced frequently and therefore their cost comes to be a regular item in the operation costs of the dragline machine.

The boom fall is practically a stationary line and seldom needs replacement, because this line is used as a running line only when the boom is being raised or lowered, which is not often. Strength

and durability are the principal considerations. The rope commonly used for this purpose, on most machines, is a 6 x 19 improved plow steel, hemp center rope. The same rope, with wire center or wire rope center, is recommended for larger machines, like a Bucyrus Class 175 or larger.

While the boom fall has a long life and its cost per unit of excavation is almost negligible, such is not the case with either the hoist line or the drag line. Working under severe conditions, they wear out fast and are subject to frequent replacement. They constitute an important item in the operation cost of the machine. Therefore, a special effort was made on this work to secure records which would be of value in selecting the best cable for these uses. Service records were kept for both drag lines and hoist lines on standard card forms, which gave the following information for each line used:

Date of Installation.
Make.
Size.
Description.
Date Replaced.
Character of Material Excavated.
Quantity of Material Handled.
Identification Number of Machine.
Names of Operators.

Many different makes and types of cable were used more or less extensively, and for experimental purposes service tests were made on many cables of special construction, supplied by various manufacturers for that purpose. Every effort was made to secure records of each line used on each of the larger machines, and the records were checked up frequently to make sure that the information obtained would be reliable. It was hoped that an analysis of such records would bring out some worthwhile information on the subject, and might lead to a type of construction which would give better service and lower cable costs on future work. The result of this effort was somewhat disappointing, in that it did not bring out any one type of rope which had an outstanding advantage over all others. A great deal was learned, however, and by no means was the effort wasted, but the field is still open, because the selection of the best wire rope for drag line purposes is still to some extent a matter of personal preference.

Even with carefully compiled records, it is difficult to make reliable comparisons of cable wear between two machines, or between two makes of cable on the same machine, the undeterminable factors are so numerous. The human equation as expressed in the skill and temperament of the operator; the care given the machine; practically undiscoverable faults, like a sheave slightly out of line on its pinion; may have more effect on cable wear than the type of machine or type of cable. This is more marked in the case of the drag cable. The character of work being done; the maximum pull of engine and motor; the diameter and type of drums; the design and construction of fair-leads; the depth of cut and the consequent bend of the line over the fair-lead sheaves; the possibility of kinks and cuts from sharp rocks, and wear from dragging through sandy material; all have a marked effect on the life of a drag cable. As a matter of fact the records obtained show clearly that a great many drag cables are not worn out at all, but are broken or cut before they hardly show signs of ordinary wear, and that the ultimate life of these lines is realized in only a comparatively few cases. While an operator is expected to use ordinary care in handling his bucket and line, his principal job is to get out the yardage, and a drag cable more or less is of small consequence compared with the steady output of the machine. In other words, while the selection of the most suitable cable for drag line purposes is worth study, and may have considerable effect on unit cost of operation, there is very little that the operator is warranted in doing to save the cable at the expense of output, outside of ordinary care, avoiding kinks, severe jerks, hard pulls when the bucket is stuck, or continual dragging over a sharp obstruction that might easily be removed.

So, in studying the many records that were obtained it has become evident that the conclusions as to the best type of cable are not as obvious as might be desired. The final analysis of these records contains only the rope used on the larger types of machines, because, since various sizes of machines were in use on this job, it was customary to use for drag lines on the smaller machines, the serviceable portions of discarded hoist lines from the larger machines.

One thing is evident, and this bears out the recommendation of manufacturers, that for both hoist and drag lines, Lang lay construction is best. Lang lay construction means that the lay

of the wires in the strand and of the strands in the rope are both in the same direction, giving increased distribution of serviceable wear and greater resistance to direct pull. In ordinary cable, known as "regular" lay, the twists of the wire in the strand, and of the strands in the cable, are in opposite directions.

Hoist Line. The hoist line passes over the sheaves on the end of the boom and at the bucket, as well as over the hoist drum, is used continuously as a running line, and therefore requires flexibility. For this purpose a hemp center rope is desirable. The hoist line is not subject to the shocks or severe service that the drag line is, but nevertheless, the continual running over the sheaves means hard service and makes the cost of replacing hoist lines a considerable item. Improved plow steel is none too good for this class of service. The standard 6 x 19 rope gives the required combination of strength and flexibility. Nothing in the records under discussion leads to any better selection for hoist lines, for machines of any size, than the standard 6 x 19, improved plow steel, hemp center, Lang lay rope. The size of rope for any size or type of machine should be taken from the manufacturer's catalogue.

Drag Line. When it comes to drag cables, the records show that two things, namely, fair-lead construction and size of drums, have a great deal to do with the life of the cable. On one machine, where extremely poor records were made as to drag line service, the length of life was increased 79% by lagging the drums, increasing the diameter from 18 to 22 inches, which was judged to be as much of an increase as the motor would stand. Unfortunately, it was not possible to do much towards improving the poor fair-lead construction on some of the machines used on this work; therefore, in selecting these records for the purpose of comparing cables, those applying to machines with poor fair-lead construction were eliminated.

As has been said, many different makes of cable were tried out, and many different types in addition to the standard types. Experiments were made on various special cables made up by different manufacturers for that purpose. These special cables were usually furnished free of charge, or with the understanding that the amount paid for them would depend on the amount of work they did. While some of these special cables seemed to have possibilities, none of them gave results that warranted their

adoption in place of the standard 6 x 19 rope in general use, and none of the uses of the special ropes went beyond the stage of experimentation. Considering that a number of rope manufacturers were enough interested in this thing to practically donate two or more lengths of cable for the purpose of experimenting, it may be assumed that the field in regard to these special cables was fairly well worked out, and that the standard 6 x 19 rope or something very similar, is at least as good as any type that has been suggested up to the present time. In trying to find something better, the effort was to increase flexibility and wearing quality without sacrificing strength. The result usually was that either strength was actually sacrificed, or that the rope did not hold its shape.

Table V gives a summary of the life of drag cables on the Class 24 Bucyrus machines engaged on earth excavation. This includes service records on 130 cables and covered a period of about four years. The three different types of cables represented here are those which gave the best results on the job, all experimental lines being eliminated. There were also eliminated, in making up the figures, 17 of these cables which failed at less than 14,000 cubic yards, it being assumed (arbitrarily) that a failure at less than 14,000 yards on these machines, and on this kind of excavation, was due to some accident which ought not to be considered in making comparisons between the cables. Such figures should be in the record however, and should be taken into consideration when arriving at the average length of life per cable throughout a job. The figures given in this table indicate that about 50,000 cubic yards is a good average life for drag cable on a Class 24 machine in earth excavation. Also that the difference between maximum and minimum records on this class of work is very great.

All these cables were Lang lay, and all were of improved plow steel. The 6 x 19 wire center was used as standard during the earlier part of the job, but later, the 6 x 19 filler wire construction, wire rope center, gave such excellent results, that it was adopted as standard during the latter part of the work. The filler wire construction is similar to the regular 6 x 19 rope, except that smaller wires occupy the spaces between the 19 larger wires in the strand, giving the rope a little more strength, but principally tending to make it hold its shape better. All the filler wire rope used on this job had an independent wire rope

Table V.—Summary of Life of 1-⅜" Drag Cables on Bucyrus Class 24 Machines, Steam and Electric on Earth Excavation.

Number of Cables	Type	Maximum Cu. Yds.	Minimum Cu. Yds.	Average Cu. Yds.	Average Number of Days
17	6 x 19 Wire Center	84,730	19,450	52,820	34
63	6 x 19 Filler Wire	165,000	14,409	50,530	21
33	6 x 25 Flattened Strand	90,820	15,000	42,580	19½

Cables Failing at Less Than 14,000 Cu. Yds. Per Cable

8	{ 6 x 19 Wire Center { 6 x 19 Filler Wire
9	6 x 25 Flattened Strand

Note: All these cables were improved plow steel, Lang lay.
The Filler Wire rope has an independent wire rope center.
The Flattened Strand rope has a hemp center.

Table VI.—Summary of Life of 1-⅜" Drag Cables on Class 175 Electric Machines on Earth Excavation

Number of Cables	Type	Maximum Cu. Yds.	Minimum Cu. Yds.	Average Cu. Yds.	Average Number of Days
3	6 x 19 Wire Center	75,100	41,300	62,100	43
18	6 x 19 Filler Wire	106,000	15,000	41,510	29
24	6 x 25 Flattened Strand	101,000	16,000	42,030	22

Cables Failing at Less Than 14,000 Cu. Yds. Per Cable.

3	{ 6 x 19 Wire Center. { 6 x 19 Filler Wire.
7	6 x 25 Flattened Strand.

Note: All these cables were improved plow steel, Lang lay.
The Filler Wire rope has an independent wire rope center.
The Flattened Strand rope has a hemp center.

Table VII.—Life of 1-⅜" Drag Cables on One Class 175 Bucyrus Dragline Machine on Earth Excavation

Number of Cables	Type	Maximum Cu. Yds.	Minimum Cu. Yds.	Average Cu. Yds.	Average Number of Days
3	6 x 19 Wire Center	75,100	41,300	62,100	43
4	6 x 19 Filler Wire	106,000	22,000	69,100	50
13	6 x 25 Flattened Strand	101,000	16,000	48,350	25

Cables Failing at Less Than 14,000 Cu. Yds. Per Cable

2 6 x 25 Flattened Strand.

Note: All these cables were improved plow steel, Lang lay.
The Filler Wire rope has an independent wire rope center.
The Flattened Strand rope has a hemp center.

center. The fact that the average of this cable falls slightly below that of the others may be due to the fact that the machines were older, and the sheaves more worn, and more often out of alignment; also perhaps because the work was speeded up more and more, as time went on, and methods improved and output increased. The men on the job were strongly in favor of filler wire construction, and it is a fact that the three highest records obtained, namely 165,000, 140,000 and 125,000 cubic yards per line, were obtained on this type of rope on the same machine. Other ropes on this machine sandwiched in between these filler wire records, gave no better service than on other machines, indicating that the higher record was due to the rope, and not to any special excellence of the machine itself or its operators. So, in spite of the fact that the records do not show any advantage in favor of filler wire construction, except that it yielded the three highest individual records, the opinion on the job is definite that this is the best type of construction so far devised for drag cable on the larger machines, and the author has no quarrel with that opinion.

Table VI gives a summary of drag cable service on the two Class 175 Bucyrus machines working on earth excavation. In this table, records on 52 cables are included, 10 of which failed at less than 14,000 yards and are eliminated in making up the figures. It should be said in this connection that one of these machines was mounted on a scow during a good portion of the period covered by this record, loading material onto other scows, which meant unusually severe conditions for the drag cable, and additional work, because it was used frequently to pull the empty or loaded scows into desired positions, necessitating throwing of the bucket with corresponding jerks on the line. For that reason Table VII is presented giving a corresponding record for drag lines used on one of these Class 175 machines, which was working most of the time on straight excavating and casting. It will be noted that the same size of drag cable gave longer service on the Class 175 machines than on the Class 24 machines. The same size buckets, 4½ to 5-yd., were used on all these machines, with a few exceptions, but the larger hoist drums on the Class 175 machines meant less strain on the drag cable.

On rock excavation the wear is much more severe and the chances of accidental breaking of drag cables are much greater. Table VIII shows a record of 20 drag cables used in rock excava-

Table VIII.—Life of 1-⅜" Drag Cables on Class M Lidgerwood Dragline Machine on Rock Excavation

Number of Cables	Type	Maximum Cu .Yds.	Minimum Cu. Yds.	Average Cu. Yds.	Average Number of Days
10	6 x 19 Wire Center	8,900	4,800	6.500	9
10	6 x 25 Flattened Strand	9,000	3,000	6,900	10

Note: All these cables were improved plow steel, Lang lay.
The Filler Wire rope has an independent wire rope center.
The Flattened Strand rope has a hemp center.

Table IX.— Comparison of Life of 1-⅜" Drag Cables on a Class 24 Bucyrus Electric Dragline Machine at Huffman Dam on Earth and on Rock Excavation

Number of Cables	Type	Maximum Cu. Yds.	Minimum Cu. Yds.	Average Cu. Yds.	Average Number of Days
	EARTH EXCAVATION				
3	6 x 19 Filler Wire	165,000	125,000	143,300	56
8	6 x 25 Flattened Strand	75,000	15,000	49,400	26
	ROCK EXCAVATION				
3	6 x 19 Wire Center	17,000	9,000	14,000	9
2	6 x 25 Flattened Strand	9,000	8,000	8,500	13

Note: All these cables were improved plow steel, Lang lay.
The Filler Wire rope has an independent wire rope center.
The Flattened Strand rope has a hemp center.

tion by a Lidgerwood Class M machine. This rock was alternate layers of limestone and shale. The cut was deep, reaching a maximum of 63 feet below the level of machine (which is considerably more than the maximum depth given in manufacturers' tables), this condition increasing the strain on the drag cable and making things still more unfavorable than they would have been otherwise. But it is apparent that a drag cable in rock excavation will last not more than 15% to 20% as long as it will in earth. It is interesting to note that not one of the 20 cables included in Table VIII gives a life of more than 9,000 cubic yards.

The highest records on the life of drag cable were obtained at the Huffman Dam. Table IX shows a record of the Class 24 Bucyrus machine at that point, on 11 cables in earth excavation and 5 in rock excavation. The earth excavation was river gravel

FIG. 17—ROCK EXCAVATION AT HUFFMAN DAM

At this point the dragline machine accomplished the unusual feat of moving 25,000 cubic yards of rock from one setting, all of which was loaded into cars and hauled away. The deepest of the excavation was nearly 60 feet below the base of the machine.

and sand, with an overburden of loam and silt. The rock was alternate layers of limestone and shale. All this material was loaded into cars. Fig. 17 shows the rock excavation in progress.

Care of Wire Rope. A brief review of the District's practice in the handling and care of wire rope may be of value in suggesting ways of prolonging the life of these lines in service. Special care was taken in handling the cables at the central warehouse, that they might be delivered to machines in first-class condition. Cable was received from the manufacturers usually on spools; sometimes short pieces came in coils. In either case, before being cut for use it was rolled out straight, avoiding kinks, and measured to the length required. Before cutting, the cable was clamped on each side of the place to be cut, and then burnt off with torches. This prevented raveling of the strands or wire, and the slipping of either in the rope. All cable was kept clean and well oiled to prevent rusting. A drag cable wears most rapidly at the end next to the bucket. Where the cable length permitted, this worn part was cut off and the cable was turned end for end to secure maximum wear. But extra long cables could not always be used to advantage, as it was found desirable not to have the line so long as to require more than one layer on the drum when the bucket was pulled in close to the machine. Some machines are not designed for deep digging with that limitation in mind, and sometimes it cannot be avoided, but it always should be avoided if possible. Pulling and crowding the top layer over and into the lower one, under the strain of loading a bucket, results in undue wear and tear on the line. It was found also that a grooved drum, scored to the diameter of the cable, materially increased the life of the cable by preventing rubbing together of the coils. Oversize cable does not give results that might be expected, at least without complete change of sheaves and a suitable modification of drums.

Cable Costs. The following figures show the influence of cable cost on the cost of operation of dragline machines. The figures cover the service record on a number of cables in each case, and on separate machines, and include all cable used, regardless of premature failure.

Costs of Drag Cable on Class 24 Bucyrus Machines.
(Earth Excavation)

Description of cable—1⅜-in. diameter, 6 x 19 improved plow steel, Lang lay, filler wire construction, independent wire rope center.

Number of cables	90	
Average length	169	feet
Cost per foot	53.9	cents
Cost per cable	$91.09	
Average life	44,300	cu. yds.
Average cost of cable per 1,000 cu. yds.	$2.06	
Average length of life	21½	days

A similar record for hoist lines on several Bucyrus Class 24 machines follows:

Costs of Hoist Cable on Class 24 Bucyrus Machines.

Description of cable—¾-in. diameter, 6 x 19 improved plow steel, Lang lay, hemp center.

Number of cables	28	
Average length	395	feet
Cost per foot	18.6	cents
Cost per cable	$73.27	
Average life	60,000	cu. yds.
Average cost of cable per 1,000 cu. yds.	$1.22	
Average length of life	34	days

The average life of these cables as shown in the foregoing, namely 44,000 cubic yards for drag lines and 60,000 cubic yards for hoist lines, are fair averages to use in estimating cable requirements for work of this sort on a long job. The average life of a hoist line may be taken to be roughly about 50% longer than that of a drag line. The average cost of a hoist line per cubic yard of excavation is approximately one-half that of a drag line. The cost of wire rope equipment is a large enough item in the cost of dragline excavation so that it is a matter of economy to secure the best rope possible for the purpose.

Special Features of Operation

Certain details of performance which were observed from time to time during the progress of the District's work are presented here for what they are worth.

Accuracy of Cutting to Grade. This depends, of course, upon the skill of the operator. For dry work, at Dayton, a heavy drag

built of timbers and old rails was swung from the bucket and pulled over the surface by the dragline after the actual excavation had been performed. A surface so finished was expected to show no variation greater than six inches from the proposed grade line. Fig. 18 is a drawing of the drag used.

When digging under water, if the water level remained stationary, the cut was expected to vary not more than a foot from established grades. An operator was able to gauge the depth below water by various distinguishable features on the bail chains of the bucket, etc. When the water level was fluctuating, it was, of course, difficult for the operators to properly compensate for the varying depth, and less accurate work resulted. Divergencies of two feet were frequent, but on the whole, cross-sections taken in the wake of the machines showed surprisingly good results. There seemed to be a tendency for the operators, when excavating below water, to dig a consistent overdepth of from six to twelve inches. This may have been due to the fact that on dragging in, the bucket is tilted with the teeth downward for a considerable distance before straightening off level, in which latter position the operator can gauge his cut more accurately. This overdepth was overcome by lessening the cut 6 inches on the instruction sheets given the operators.

In excavating rock at the outlet structures it was found impracticable to dig closer than a foot to grade, as the bucket teeth would pull and shatter the limestone layers some distance back from the line of excavation. Finishing was done by jap drills, light blasting charges and hand work. In excavating at the bottom of the pit, it was necessary to leave the first cut several feet high, so that repeated drags of the bucket in picking up material shot down later, would not tear out the floor several feet below grade.

Deep Cuts. Mention has previously been made of the adaptability of the dragline for making deep cuts. An extreme condition was met at Taylorsville where a Class M Lidgerwood electric machine with 100-foot boom, successfully excavated in the pit of the outlet structure, 63 feet below and 37 feet above the machine base. It probably would have been possible to dig to still greater depth, limited only by the length of drag cable that could be held on the drum. However, the economic depth, limited by the excessive cable wear, and time consumed in lifting the bucket, was probably reached on that work. The fact that it was a rock cut,

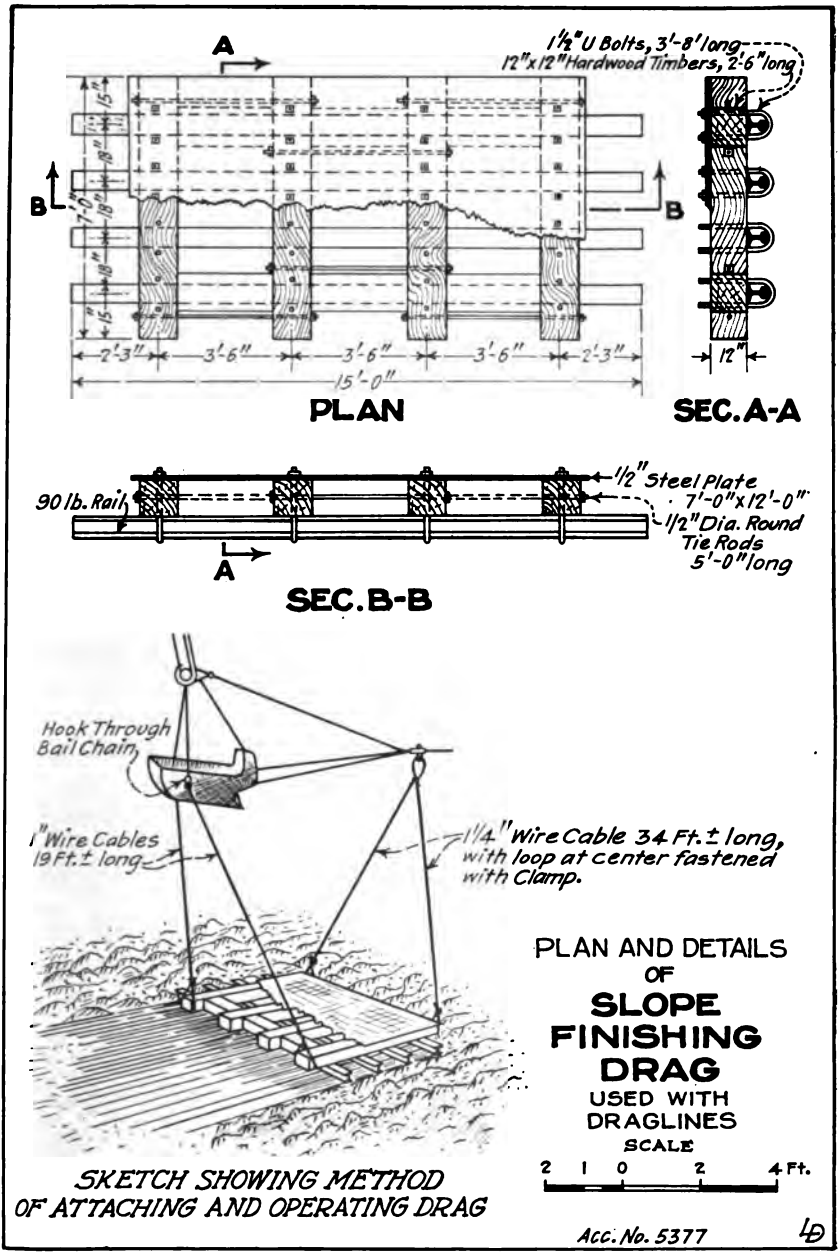


FIG. 18—DRAG FOR FINISHING CUTS

Cuts along the river channel were finished by suspending this drag from the bucket and pulling it over the surface, following the rough excavation.

made it possible for the machine to stand closer to the edge, and carry down a steeper slope than could be done in an earth cut. Also, rock excavation, due to its higher unit costs, is affected in less proportion by slowing up of hoisting and dumping operations than is earth excavation.

At Huffman dam a Class 24 Bucyrus machine, 100-foot boom, 4½-yd. bucket, made a cut 58 feet below the ground on which the machine stood, and 64 feet below the loading track. Incidentally, this machine took out 25,000 cu. yds. of work, without moving from one position—an unusual performance. Fig. 17 shows the machine at work in that cut.

Throwing the Bucket. The width of cut for the machines on the various runs was usually determined within the limit of reach of the bucket, when hanging directly under the end of the boom. However, it was sometimes desirable to reach out farther and pick up material that had slumped off or washed into the edge of the previous cut. An experienced operator was able to accomplish this by drawing in on his drag cable with the bucket suspended, then releasing the cable and allowing the bucket to swing out beyond the end of the boom before dropping. This process is known as "throwing the bucket." Observations taken at Hamilton while excavating from the channel showed the possibility of throwing the bucket the following distances beyond the end of the boom:

Class 24 Bucyrus, 100-foot boom, 4½-cu. yd. bucket....35 feet.
Class 14 Bucyrus, 60-foot boom, 1½-cu. yd. bucket....20 feet.
Model 21 Marion, 35-foot boom, ¾-cu. yd. bucket....10 feet.

Time of Swing. The time required for making a complete operation—digging, swinging and dumping—varies with the nature of the material excavated, the angle of swing, depth of cut, height of dump, care necessary in dumping, etc.

A Bucyrus Class 175 machine can dig under water, swing through an arc of 180 degrees and dump into a spoil bank, in 50 or 55 seconds, if the digging is in fairly loose gravel. The same machine, digging loose, dry material and swinging 45 to 90 degrees, can complete the operation in 34 to 36 seconds.

At Huffman, a Bucyrus Class 24 electric machine digging from the borrow pit and loading 12-cubic yard dump cars, averaged for the round trip from 52 to 65 seconds, depending on the angle of the swing.

Rate of Travel. It was not often that the machines were required to travel any considerable distance on deadhead trips, but the following data was obtained.

At Dayton a Class 175 Bucyrus, truck mounted, traveled 300 feet in five hours. This is at the rate of 60 feet per hour and means an average of 15 minutes for shifting a set of mats ahead and moving forward over them. The trip was made over ground that had been previously prepared and no dirt moving was required.

At Hamilton a Class 24 Bucyrus on skids and rollers moved 3,000 feet along the river bottom in nine shifts. This was at the rate of about 35 feet per hour.

At Huffman the average time for moving a Class 24 Bucyrus machine on skids and rollers over a distance of 32 feet, and chucking up ready for excavation again, was about 40 minutes.

A Class 14 Bucyrus on skids and rollers moved about 6,000 feet in twenty shifts, an average of about 300 feet per 10-hour shift, very little track grading being necessary.

A Class 14 Bucyrus on caterpillars moved 5,800 feet over city streets in six 10-hour shifts, or nearly 1,000 feet per shift. On this trip there was a delay because of a broken caterpillar coupling, and there were trolley and telephone wires in the way which delayed progress.

This same Class 14 Bucyrus on caterpillars moved 5,000 feet over city streets in about three 10-hour shifts, where moving conditions were more favorable.

A Model 21 Marion on caterpillars moved 8,000 feet over city streets in about 15 hours.

Performance on Curves. A Class 175 Bucyrus, truck mounted, traveled on a curve of 135-foot radius, figured to the center of the machine. This is about the limit for safe and economical operation.

A Class 175 Bucyrus, mounted on skids and rollers, turned 180 degrees on a 75-foot radius. It is possible that an even sharper curve might be traveled but it would not be an economical performance. Considerable difficulty was found in making the curve and 40 or 50 rollers were crushed in the operation.

With a caterpillar mounting it is possible to turn in a very short radius by switching back and forth. No special determinations of performance were made.

The mounting of the walking type machine permits movement in any direction from a given position.

Performance on Grades. The work in Dayton frequently required the machines to move up or down between considerable differences in elevation. The following data resulted from careful observation.

A Class 175 Bucyrus, truck mounted, climbed up a grade of 5.1 per cent, pulling the rear trucks over a 7½% grade at the start. This was very nearly the limit for that equipment. The machine climbed with great difficulty and some trouble was experienced in swinging with a loaded bucket.

The same machine, mounted on skids and rollers, climbed out of the river bottom on a 7% grade without much trouble.

A Class K Lidgerwood, truck mounted, traveled and operated down a 5.9% grade with very little difficulty. At another time the same machine worked down 5% and 7% grades.

A Class 9½ Bucyrus, caterpillar mounted, worked down a 13½% grade at one place. At another place it worked both up and down on a grade which averaged 13.2% for 80 feet, but which was 16.2% for half that distance. It is believed that such a grade is about the limit for the machine.

A Model 36 Marion, caterpillar mounted, climbed a 20% grade with some difficulty. This is about the limit for a machine of that size.

A Monighan, type 3-T, operated with scarcely any difficulty on a 6.8% grade. It would be safe to try steeper grades.

Passing Under Bridges. At Dayton, a Class 175 Bucyrus, passed under six bridges in the course of its construction work. Three of these passings were made while the machine was floating on a spud scow, one was a dry land passing, and two were land passings with the aid of cofferdams.

A brief description of the passing of the Class 175 Bucyrus, under Main Street bridge will be given as typical of those made with the floating equipment. The machine was mounted on a 40-foot by 80-foot spud scow. A Class 9½ Bucyrus machine, on another scow excavated downstream from and under the bridge to a depth and over an area sufficient to allow the passing of the spud scow under and out from under the bridge on the downstream side. The boom of the big machine was lowered onto one of the larger scows and the whole equipment was maneuvered into position by the steamboat. Even with the top members of

the main A-frame lowered, it was necessary to partly sink the spud scow to secure the necessary clearance. It was thought that water ballast could be pumped into the scow to secure the necessary draft, but the splurging of the water in the hold made the scow tip dangerously, and loading the deck of the scow with gravel was resorted to as a more successful method. The whole equipment was then pushed and guided under the bridge by the steamboat. The total time required by the passing was about six working days.

The passing under Fifth Street bridge was made on dry land, the machine being mounted on skids and rollers. The track upstream from the bridge was prepared by the machine itself, that under and downstream, by men and teams. The boom and A-frame were lowered, the machine pulling itself under by its drag cable and a dead-man. The track mats were dragged ahead by teams. The procedure sounds simple enough, but actually it was a slow, tedious process, about two weeks being consumed in preparatory work and actual passing.

In passing under another bridge, it was not possible to secure the necessary clearance for the top of the machine, even at low water stage. Consequently a long cofferdam was thrown out, and excavation made in the pit of sufficient depth to allow clearance under the bridge. The procedure was then similar to passing under Fifth Street bridge. The excavation work of the machine was interrupted for only two shifts in making this passing.

At Hamilton, a Class 24 Bucyrus passed under bridges, with A-frame lowered, ten times. All of these passings were made on dry land, the grading for the track being done by teams and by a Model 21 Marion dragline. Heavy timbers were used for the rollers instead of mats. A line of 60-pound steel rail was placed along the center, the lowered boom sheave wheel being supported and running on this track. The machine pulled itself through by use of its drag cable and a dead-man. The first passing required two 10-hour shifts. The result of experience and of standardizing this type of work is indicated by the fact that the last bridge passing was made in six hours.

Crossing Rivers. Many times during the course of this work it was necessary for a dragline machine to make a river crossing. The ease with which that is accomplished with a dragline machine is notable, as compared with a similar undertaking with almost any other type of excavating equipment, many of which

would have to make for the nearest bridge. It is accomplished by simply throwing out a trackway, in the form of a jetty, into the stream, running out upon it, and then, by taking up the material from the rear and placing it ahead, proceeding across on a moving island which eventually joins the shore on the other side and forms the trackway for leaving the channel. Fig. 19 shows one of the Class 24 Bucyrus machines in the act of making one of these crossings. The machine is proceeding to the right, and has just joined its moving island to the shore, forming the jetty or peninsula on which it will track out of the channel. The water

FIG. 19—DRAGLINE MACHINE CROSSING RIVER

The crossing began at the far shore by building out a peninsula as a roadway for the machine, this work being done by the machine itself. The machine then rolled out on to the peninsula, picked up the material from the rear and placed it ahead, thus forming a moving island which gradually approached the other shore. In this view the moving island has just been joined to the shore, forming a trackway on which the machine will move out of the channel, clearing up the fill behind it as it goes.

here is four or five feet deep. The important thing in making these crossings is to leave waterway enough so as not to increase velocity of current to the point where the loose material at the ends of the "island" will be washed out. Character of material available is sometimes quite a factor. In this case the material is ideal for the purpose, with a good percentage of coarse heavy gravel.

Delays and Interruptions

Interruptions in the operation of dragline machines constitute an essential part of their cost of operation. Consequently much study was given, on the various jobs, to the reduction of this to a minimum. Delays may be classified as major and minor.

Major delays include major repairs and overhauling of the machine, accidents with their resultant repair, and delays due to high water. All are more or less unavoidable in nature, but the extent of time consumed may be regulated in most instances. Even high water delays can be reduced by so planning the work as to have the machines in locations not subject to such danger during the seasons when floods may reasonably be expected. At the dams, work was generally discontinued during a few weeks of most severe winter weather, giving excellent opportunity to overhaul and make necessary repairs to the draglines without interfering with their efficiency during the operating season. On the channel work the operation of the machines was continuous, repairs being made as required during the progress of the work, and this should be kept in mind when comparing delays on the two classes of work.

Minor delays include minor machine adjustments and repairs, changing cables, moving the machine, power delays (due both to circuit shut-offs and moving of transformers), delays incidental to car loadings, etc. Although the duration of separate minor delays is short, their total for a season's work is generally greater than the total of major delays. Moreover, the human equation enters into this class of delays to a large extent, thus offering a prolific field for the promotion of more efficient machine operation, and efforts were concentrated on reduction of these delays.

At Dayton, each of the three large machines was equipped with a "Service Recorder." This is the type of recorder commonly used in connection with motor truck operation. The chart revolves by clockwork and the period of operation is recorded by a stylus attached to a pendulum so constructed as to oscillate when the machine is in motion. The recorder was fastened to the side of the dragline cab, near the rear, there being the most definite vibration of the machine at that location. Figure 20 is a specimen chart of one of the machines. Such records are, of course, valuable for reference, but further use was made of them by

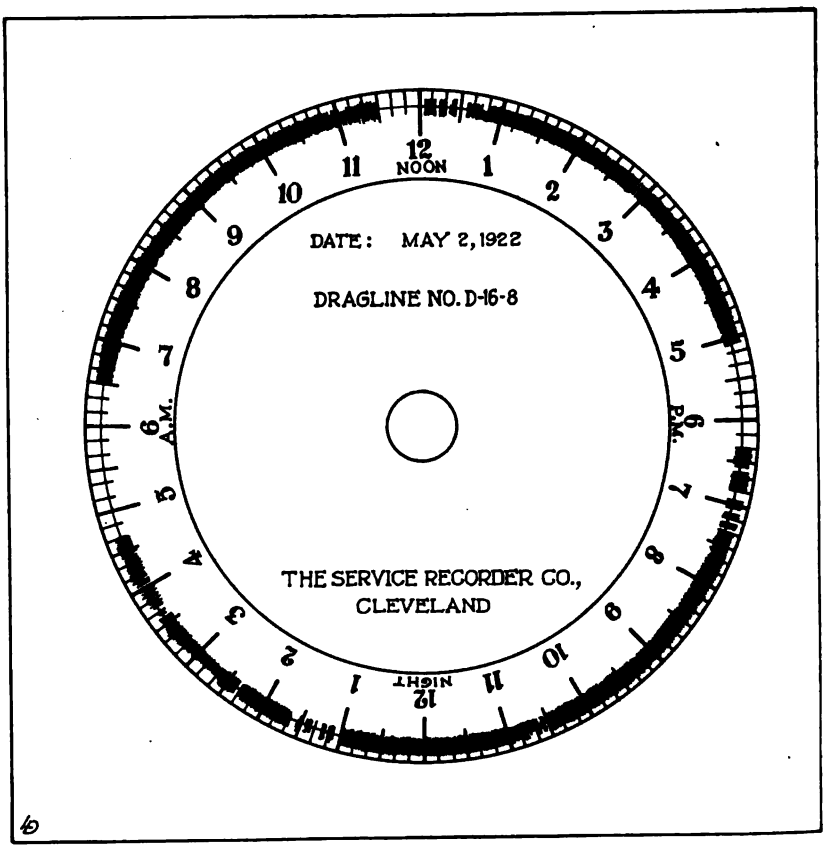


FIG. 20—SERVICE RECORDER RECORD

The service recorder is a type commonly used in connection with motor truck operations. It was adapted to use on dragline machines and gave a positive record, inaccessible to the operator until released, showing exactly the periods during the shift when the machine was in operation and when it was idle. For example, this card shows the morning shift starting at 6:30 A. M. with the work uninterrupted until the regular shut-down at 11:30. There was intermittent work from 12 until 12:40, and then continuous work until 5 o'clock with a slight interruption for a few minutes at 2:50, and so on for the rest of the day. All the interruptions indicated on the chart are fully explained in the dragline runners' report.

making the results known to the operators from week to week and thus stimulating a friendly rivalry between the several crews. It is believed that the amount of work done was materially increased as a result. It was not uncommon to see part of a crew working during the noon hour on cable changes or minor repairs.

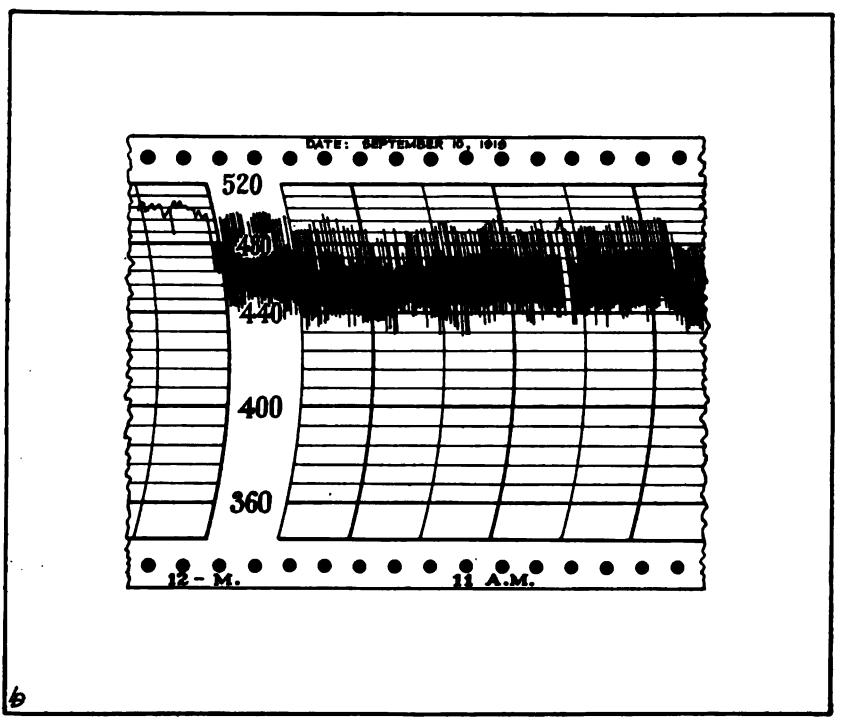


FIG. 21—VOLTMETER CHART OF DRAGLINE OPERATION

This instrument gives the same information as that described under Fig. 20. In this case there was only one electrically operated machine on the job and the voltmeter was placed in the office of the superintendent, giving him positive information during every hour of the day whenever the operation of the machine was interrupted from any cause. This portion of the chart covers only about 2 hours' running. Reading from right to left it shows uninterrupted work from about 10:15 until the regular shutdown at noon except for a few moments at about 10:50. This chart has the advantage of showing also each separate cycle of operation.

At Hamilton, where only one electrically operated machine was used, a voltmeter was located in the office of the construction superintendent. This instrument indicated at all times whether

the machine was in operation. Not only that, but it furnished a graphical record of every swing of the machine, so that it was possible from this voltmeter chart, to analyze the performance of the machine over long or short periods. Most "time of swing" records are made over a short period, and very often are recorded only for work under favorable circumstances. At Hamilton, however, such records were available not only for short favorable periods, but the record was unbroken, month after month. Fig. 21 is a reproduction of a short trip of this voltmeter record, covering a period of about two hours.

Analysis was made of the performance of the Class 24 Bucyrus machine at Hamilton during the entire month of September, 1919. The total possible working time during this month was 500 hours, of which 431 hours and 30 minutes were spent in actual digging, the other time being spent in moving, waiting for cars, minor repairs, etc. The maximum number of swings per hour during this month was 82, an average of one swing in 44 seconds; but for one day the highest number of swings was 1,332, or an average of one swing in 54 seconds; and for the whole month the number of swings was 25,543, or an average of one swing in 1 minute and 12 seconds.

The maximum yardage for one day was 4,350, while the average for the month was 2,900 cu. yds. per day. These figures are given simply to emphasize the difference between maximum performance for a short time, and average performance over long periods.

At Englewood the total of car loadings for each machine was posted daily where all concerned might see, and a very lively interest and rivalry among the crews was thus instigated. Table X gives the relation of possible operating time to actual operating time for the steam and electric machines, for the season of 1920, by months. Table XI classifies dragline delays during a period of three months of 1921. This classification is fairly representative of what may be expected on such machines, loading cars from a borrow pit in river gravel and sand.

At Huffman, only one dragline was used in the borrow pit, and consequently the opportunity for rivalry among crews was less than at other jobs. Table XII gives the ratio of possible to actual time during the season of 1921, with a classification of delays for the same period given in Table XIII.

Table X.—Dragline Performance—Englewood Dam
Season of 1920

		Total Working Time, Hrs.	Actual Digging Time, Hrs.	Time Lost Hrs.	Percent of Delays	No. Cars	Cu. Yds.
Steam Dragline	April	490	466.75	23.25	4.7	5,666	48,000
	May	500	491.42	8.58	1.7	8,396	71,000
	June	520	499.67	20.33	4.0	9,142	78,000
	July	520	479.50	40.50	7.8	9,909	84,000
	Aug.	520	463.17	56.83	10.9	9,270	79,000
	Sept.	500	455.33	44.67	8.9	8,948	76,000
	Oct.	400	356.91	43.09	10.8	6,759	57,000
	Nov.	470	401.91	68.09	14.5	7,035	60,000
	Dec.	340	320.33	19.67	5.8	5,452	46,000
	Totals	4,260	3,934.99	325.01	7.6	70,577	599,000
Electric Dragline	April	460	438.67	21.33	4.6	6,325	54,000
	May	500	476.41	23.59	4.7	8,578	73,000
	June	520	488.60	31.40	6.1	9,016	77,000
	July	520	425.75	94.25	18.1	8,085	69,000
	Aug.	520	484.60	35.40	6.8	8,904	76,000
	Sept.	500	454.00	46.00	9.2	8,547	72,000
	Oct.	390	359.25	30.75	7.9	5,700	48,000
	Nov.	480	446.60	33.40	6.9	7,798	66,000
	Dec.	340	332.41	7.59	2.2	6,188	52,000
	Totals	4,230	3,906.29	323.71	7.7	69,141	587,000

Comparison of Steam and Electric Machines

At the Englewood Dam there was an exceptional opportunity to make a direct comparison between two Class 24 Bucyrus dragline machines, one steam and one electric, which were used at the borrow pit to excavate material for the dam embankment. This material was loaded into cars, by which it was transported to the dredge pump sumps. The two machines worked side by side for about three years. They were of the same size and make, and were excavating the same kind of material under identical conditions. A close watch was kept on these operations, and for several months field costs were recorded for the purpose of comparison.

The borrow pit occupied the broad, flat valley of the Stillwater River for about three-fourths of a mile immediately above the damsite. The draglines were placed in tandem along one edge of the pit, parallel to which were laid tracks on which the loaded

Table XI.—Analysis of Dragline Delays—Englewood Dam
July to September, 1921

	July				August				September			
	Electric		Steam		Electric		Steam		Electric		Steam	
	Hrs.	%	Hrs.	%	Hrs.	%	Hrs.	%	Hrs.	%	Hrs.	%
Possible Working Time....	485	100	485	100	540	100	540	100	500	100	500	100
Actual Time Digging.....	434.5	89.6	420.4	86.7	472.25	87.5	510.3	94.5	408.3	81.7	419.5	83.9
Electrical Delays.....	3.75	0.8			5.3	1.0			11.6	2.3		
Mechanical Delays.....	27.25	5.6	37.6	7.7	23.9	4.4	16.2	3.0	7.7	1.5	49.0	9.8
Waiting for Cars.....	20.42	4.2	27.0	5.6	38.5	7.1	13.5	2.5	72.4	14.5	31.5	6.3
Total Delays.....	51.5	10.6	64.6	13.3	67.7	12.5	29.7	5.5	91.7	18.3	80.5	16.1
Cu. Yds., Day Shift.....	37,500		31,900		43,700		41,200		32,600		33,000	
Cu. Yds. Night Shift.....	34,200		29,000		41,300		36,800		29,700		29,000	
Total Yardage.....	71,700		60,900		85,000		78,000		62,300		62,000	
Cu. Yds. per Running Hr.	165		145		180		153		152		148	

Table XII.—Dragline Performance—Huffman Dam
Season of 1921
(Class 24, Bucyrus Electric Machine)

	Cu. Yds. Excavated in Month	Possible Digging Time Including Delays, Hours	Cu. Yds. Per Hour Including Delays	Actual Digging Time, Hours	Cu. Yds. Per Hour Actual Digging Time
January	60,440	500.00	121	370.35	163
February	20,940	295.75	71	199.79	105
March	62,130	470.00	132	361.66	171
April	66,840	520.00	129	444.42	151
May	70,610	500.00	141	437.92	161
June	60,660	527.17	115	386.62	157
July	53,630	475.30	113	406.56	132
August	60,190	540.42	111	435.57	138
September	50,000	477.46	105	367.34	136
October	50,560	529.63	96	395.65	128
November	43,450	464.50	94	346.07	125
Total	599,450	5,300.23		4,151.95	
Season Average			113		144

Table XIII.—Analysis of Dragline Delays—Huffman Dam
Season of 1921

	Hours	Percent. of Total Delays	Percent. of Total Possible Running Time
Waiting for Cars	660	57.5	12.5
Cleaning Loading Tracks	19	1.7	0.4
Moving Machine on Mats	171	14.9	3.2
Changing Drag and Hoist Cables	27	2.3	0.5
Mechanical Repairs and Adjustments	186	16.2	3.5
Electrical Trouble and Repairs	45	3.9	0.8
Power Off	33	2.9	0.6
Unclassified Delays	7	0.6	0.1
Total Delays	1,148	100.0	21.6
Total Actual Digging Time	4,152		78.4
Total Possible Digging Time	5,300		100.0

cars were hauled away. The steam machine worked the north half of the pit and the electric the south half, taking successive cuts from along the pit edge and dumping the excavated material into the cars alongside. This material was remarkably uniform, consisting of about two feet of light loamy top soil underlaid with sand and gravel. The total depth of the pit was about 15 feet, about 3 or 4 feet of which was below water surface.

Each machine was equipped with an 85-foot boom and a 4½-yd. bucket. In working condition the electric machine weighed about 120 tons and the steam about 140 tons. They both came onto the work new, and were in practically continuous use for three years except for a few weeks during the winters. The work was heavy and unremittent through two 10-hour shifts, 20 hours a day, six days a week, so the records given are not for picked machines under ideal conditions, but rather for hard-worked machines in actual service.

The steam machine had a main engine with double cylinders 10½-in. x 12-in., and a swinging engine with double cylinders 8-in. x 8-in. The boiler was 58-in. x 14-ft. and carried steam at 125 to 150 pounds. Coal was switched from the main line of the railway about a mile away and was unloaded by clamshell on the ground near the damsite. A team carted the coal to the vicinity of the dragline whence it was carried on to the machine by hand. Water for the boiler came from a steam pump drawing its supply from the borrow pit pool.

On the electric machine the main motor (handling the drag and hoist drums) was 200-H.P., and the swinging motor was 75-H.P. These motors were of the slip ring, variable speed type. The power was taken from a transmission line connected with a 33,000 volt circuit of the Dayton Power & Light Company. A transformer station at the dam stepped this current down to 2,300 volts for distribution to the camp and the work. The machine used current at 440 volts from three 75-K.W. transformers connected to it by flexible cables, and moved along from time to time as the machine progressed with its work. Current was 3-phase, 60-cycle, alternating.

Table XIV shows the costs of operation for a four months period under five headings—Labor, Power, Supplies and Minor Repairs, Cable and Depreciation. Minor repairs are combined with supplies, for the reason that the charge to this item is practically all for material. The labor which might properly be

Table XIV.—Comparison of Operation Costs
Steam and Electric Draglines—Englewood Borrow Pit
May to August, Inclusive, 1921

	Total		Per Cu. Yd.		Percent of Cost	
	Electric	Steam	Electric	Steam	Electric	Steam
Labor	$6,926	$11,185	$0.0245	$0.0404	38.1	53.6
Power	4,492	4,242	.0159	.0153	24.6	20.3
Supplies and Minor Repairs	1,993	1,821	.0070	.0066	11.0	8.7
Cable	1,685	844	.0060	.0031	9.3	4.1
Depreciation	3,092	2,774	.0109	.0100	17.0	13.3
Totals	$18,188	$20,866	$0.0643	$0.0754	100	100
	Cu. Yds.	Cu. Yds.				
Total Work	283,000	277,000				

charged to minor repairs was performed by the regular crew, and is therefore included under the "Labor" heading. The use of cable on dragline machines is an important item and is therefore set out in this account. Depreciation is estimated at 2% per month, of the value of the machine, and covers interest on investment as well as straight depreciation. It may be said that this charge of 2% per month has worked out about right in case of the many machines of this type which have been disposed of after their work was completed. It has usually left a little balance in the plant account on the credit side.

It should be carefully noted at the outset that the figures given in Table XIV are only for ordinary running expenses and depreciation. They do not include cost of track construction and maintenance, nor construction and maintenance of electric power lines; neither are overhead charges included. In other words, they are only field costs of operation. As these figures were made up for the purpose of comparison of the two machines, it would add nothing of value to attempt to include all general items, some of which would have to be estimated anyway. The items which are not included would balance each other approximately as regards the two machines, and it is felt that the comparison is easier to follow through without them.

"Labor" is much the larger item in both accounts, being approximately 38% of the total for the electric machine and approximately 53% for the steam machine. It is also the item in which the electric machine shows its chief saving, its total labor cost

Table XV.—Comparison of Crews
Steam and Electric Draglines—Englewood Borrow Pit
May to August, Inclusive, 1921

Crew	Electric	Steam
Runner, @ $278 per month	1	1
Oiler, @ 54c-56c per hour	1	1
Fireman, @ 53½c per hour	..	1
Pumpman, @ 50c per hour	..	1
Pit Boss, @ 52c per hour	1	1
Ground Men, @ 40c per hour	2	3
*Team and Man, @ 88c per hour	..	1
Total	5	9 and team

*Day shift only.

Table XVI.—Cost of Electric Power
Class 24, Bucyrus Dragline at Englewood
May to August, 1921

Month	Maximum Demand K.W.	Consumption K.W. Hours	Cost	Cost Per K.W. Hour
May	1,404	51,290	$ 968.83	$0.0189
June	1,440	47,090	918.25	0.0194
July	1,362	69,600	1,259.96	0,0181
August	1,404	74,000	1,344.78	0.0182

for the period being $6,926, against $11,185 for the steam machine. These figures include all ordinary labor around the machines, that of the pumpman who attends to the pump and boiler which supplies feed water to the steam machines, and also that of the team and man for hauling coal. The pumpman is required on both the day and night shift, but the coal team works on the day shift only. It is these two items, together with the wages of fireman and extra man required to help handle the coal from the pile on the ground to the machine, which run up the labor cost on a steam machine. In many cases an oiler is not required on a steam machine in addition to a fireman, but in a case like that at Englewood, where the machine is working steadily at full capacity, it is found economical to have an extra man there. As further illustrating the difference in labor cost of the two machines, Table XV is presented which shows the itemized cost of the two crews.

The "Power" item includes electric energy delivered to the job in one case, and coal delivered to the main distributing point on the job in the other. The charge for electric power depended not only on the actual amount of current used, but also on the "maximum demand" for energy on the entire work during the month. Table XVI illustrates how this charge varied from month to month. It is of interest in this connection to call attention to the small loss of time due to interruptions to electric service. For the total four months period, 2,080 working hours, there were only four shut-downs, totaling nine hours in all, or less than one-half of one per cent of the working time during the period.

The cost of coal for the steam machine, delivered at the distributing point on the job, not including team haul to the machine, averaged $6.52 per ton. The total amount used during the period covered by this comparison was 571 tons. The cost of hauling this coal to the machine during the four months period totaled $906.40, this item being included in the labor account as already shown.

Further examination of the figures in Table XIV will show that the cost per cubic yard is least with the electric machine, averaging 6.43 cents per yard for the period, as against 7.53 cents for the steam machine, an advantage of about 15% in favor of the electric machine. During the period the electric machine handled 283,000 cu. yds. and the steam machine 277,000 cu. yds. It should be made clear that the differences in yardage or costs were not caused by a difference in the ability of the operators. (There were two for each machine, one each for each shift.) All were first-class men, able to handle either electric or steam machines, and there was little to choose between them.

As to speed *while in operation,* the steam machine seemed to be somewhat faster, due to its quicker swing and hoist. There is more elasticity in a steam engine than in an electric motor, and therefore somewhat smoother control. But the superiority of a steam machine while in operation, is more than offset by the loss in shut-downs for minor repairs, especially at places like this job where the water is so hard on boilers. There is no question about the superiority of the electric machine for continuous operation under such conditions. It might seem from a comparison of the figures for the costs of minor repairs, that they do not

bear out this conclusion, but it should be remembered that these costs (for minor repairs) as shown in these tables, include material costs only, the labor on repairs being performed by the regular crews, and included in the "Labor" item as explained heretofore. Therefore the difference between the two machines in this particular, is shown in loss of time, and cost of output, rather than in actual cost of repairs.

As to the cost of power, note that the finished product, electric current, is put on to a distribution line at the dam at only slightly higher cost than the raw product, coal, is delivered to a storage pile on the job; the figures being $4,491 against $4,242 for the total period. The first moved 283,000 yards and the second 277,000 yards. Remember also that a good part of the excess cost of steam operation lies in the labor of getting the raw material, coal, handled by men and delivered from the storage pile on to the dragline and under the boiler. Some of the remaining excess lies, very likely, in the crude mechanism of engine and boiler as a means of transforming coal into mechanical energy at the drag cable, as compared to the higher efficiency of the several transformations in the electrical system from the high tension power line to the working parts of the machine.

One other thing that should be mentioned especially is the charge for cable. This comparison shows a much higher charge on the electric machine, the total expense for this item during the four months period being $1,685 against $844 for the steam machine. It may be that this difference is more than it would be over a longer period. In fact, a record on 30 or more drag cables each, for these same machines, shows a difference of about 10% in favor of the steam machine. Undoubtedly, this is due to greater elasticity in the operation of the steam machine, leading to less shock, less strain, and therefore longer life of the cable. The greater difference which appears in this four months record, is probably due to a chance piling up of cable expense on the electrical machine for this particular period, since replacements of cables occur at irregular intervals.

The result of this comparison, showing as it does a decisive advantage in favor of the electrical machine, is borne out by the experience on the work of the District as a whole. The electric machine has been found to be more economical, less troublesome, cleaner and more comfortable for the crews—and it is recognized

that more comfortable working conditions lead to more efficient workmen. The electric machine is more constantly reliable, freer from breakdowns and delays, and in general more efficient.

CHAPTER III.—CONSTRUCTION PLANT

This chapter is given over largely to lists of construction plant and equipment used on the various jobs, including a brief description of each piece of equipment together with its cost. The equipment is listed under sub-headings, representing the principal classes of work on each job. For a large portion of this equipment, reasons for the selection will be self-apparent. In those cases where a choice of plant or methods was open, or where the reason for the selection is not apparent, an attempt will be made to discuss the more important considerations leading up to the choice. The equipment is listed by jobs, for the reason that each job was complete in itself—practically an independent unit, except for the centralization of general service, certain clerical work and general supervision.

Generally speaking, the total of these lists covers all of the Conservancy equipment in use during the construction period. In some cases, however, the same piece of equipment is listed under two or more jobs—if it was transferred from one job to another—so there are some duplications. On the other hand, a few of the jobs are not mentioned at all—those few on which no important equipment was used, other than that included in these lists, and where methods and results were the same as those described hereinafter.

Germantown Dam

Description. The Germantown Dam is located on Twin Creek, a tributary of the Miami River, at a favorable site about two miles northwest of the Village of Germantown. The valley at that place is narrow, with steep side hills, forming a good dam site and permitting the construction of a high dam with comparatively small yardage.

The dam is an earth embankment, with a pair of concrete outlet conduits at the elevation of the old river bed. The total volume of the embankment is about 835,000 cu. yds., of which 800,000 cu. yds. were placed by the hydraulic fill method. The remaining 35,000 cu. yds. was that obtained from the conduit ex-

cavation and the cut-off trench, and was placed in the dam by other methods. The dam has a maximum height of 114 feet, and measures 1200 feet along its crest, which is 30 feet wide and carries a roadway across the valley. Both the upstream and downstream slopes vary from 2 to 1 at the top to about 4 to 1 at the bottom, with 10-foot berms, the first berm 20 feet below the top and the others 30 feet apart. Across the old river channel the side slopes tail out to about 10 to 1, both upstream and downstream, forming a blanket over the exposed river bottom. Fig. 22 shows a typical cross section of the Germantown Dam, and this is typical of the other dams, being modified only as to height.

The outlet conduits were located at one side of the valley floor, at the foot of the side hill, and some distance away from the river channel, to which they were connected by new channels.

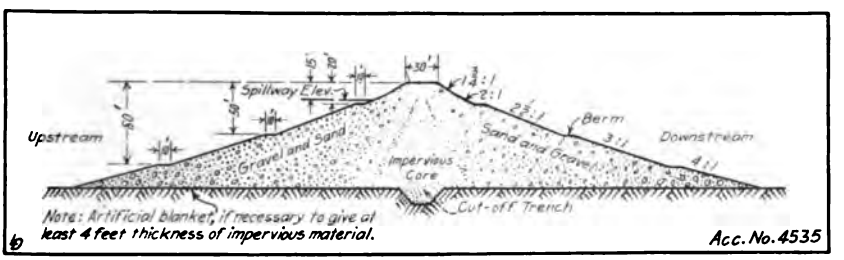


FIG. 22—TYPICAL CROSS SECTION OF DAMS

Thus they could be built complete, without disturbing the river channel. The flow could then be diverted through them whenever desirable. The conduits are horse-shoe shape, about 13 feet wide and 9 feet high, although for stream control purposes during construction, they were first built about 14 feet deeper, the extra depth being filled in afterwards. They are 546 feet long from head wall to head wall, with concrete wing walls at the inlet end, and a concrete stilling pool at the outlet end. These outlet works contain in all 16,350 cu. yds. of concrete. The spillway is a concrete lined gap, 76 feet wide at the bottom, through a depression in the hills about 800 feet north of the dam. A reinforced concrete girder bridge with three 40-foot spans and a short span at each end, carries a roadway across the spillway cut. The spillway concrete, including the bridge, amounts to 950 cu. yds. Fig. 23 shows the general layout of this job.

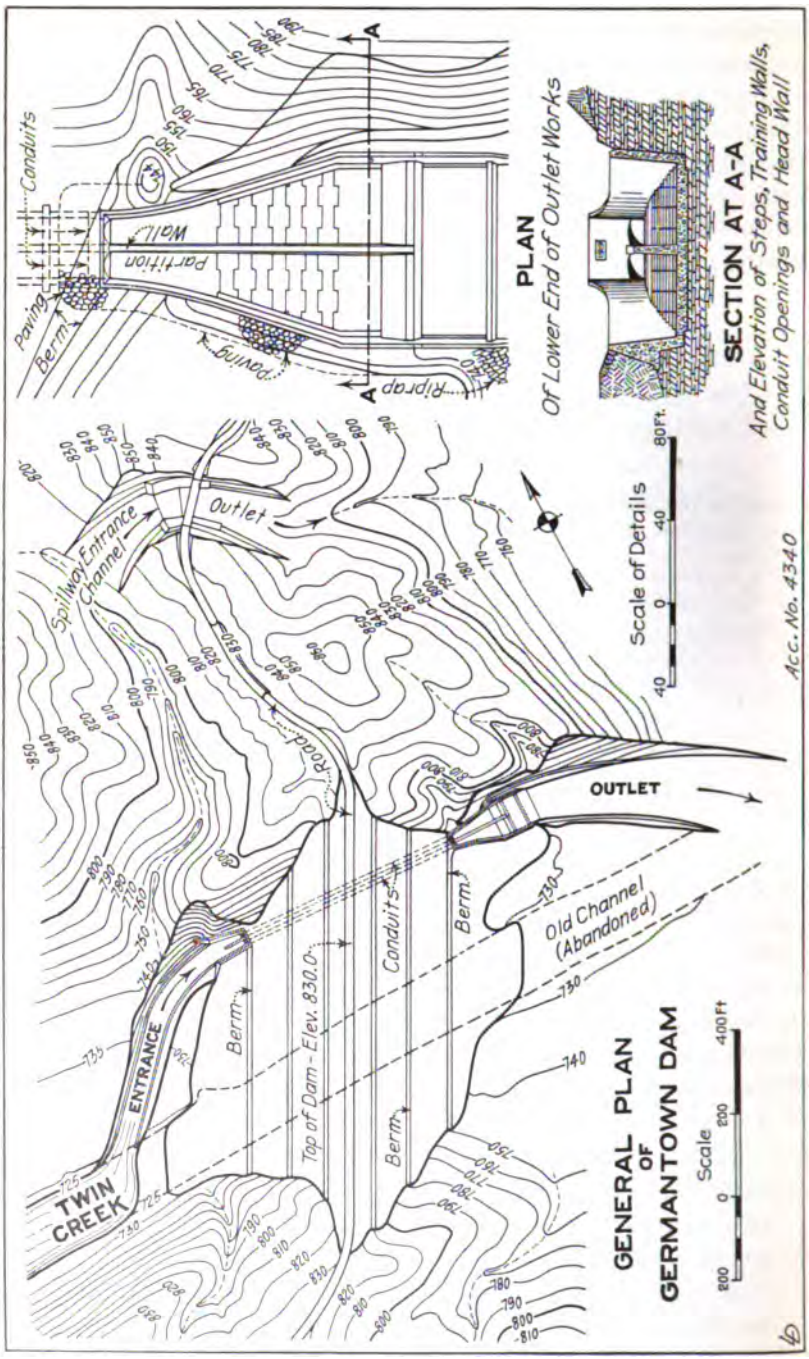


FIG. 28—GENERAL MAP GERMANTOWN DAM
Showing general relation of dam, outlet conduits and spillway.

Railroad Facilities. The nearest railroad was at Germantown, which became the receiving point for all machinery and equipment as well as for cement, coal and other supplies. From there everything was hauled to the job, a distance of about 3½ miles over hilly country roads. This transportation was an item which had to be taken into consideration in selecting the equipment for the job, and had its effect on costs of work throughout. Some of the difficulties of getting the equipment in to the dam are illustrated in Fig. 24, where a construction train is being moved

FIG. 24—CONSTRUCTION TRAIN MOVING IN TO GERMANTOWN DAMSITE

This construction equipment moved under its own power to the damsite from the nearest railroad, a distance of about 3½ miles over hilly country roads. Sections of track released in the rear were hauled forward and relaid ahead.

under its own power on a track laid a section at a time, over village streets and country roads. Sections of track released in the rear were hauled forward and relaid ahead. Where grades were too steep for the train as a whole, a siding was built on top of the hill, the front engine pulled up the cars one or two at a time, and the train was made up again at the top of the grade. Other equipment, dismantled parts, supplies, etc., were hauled by truck with or without trailer. Truck hauling was an important feature of this job.

Construction Program. The construction program at Germantown, as well as at the other dams, centered around the question of stream control during construction. There must be no possibility of flooding the communities below the dams, due to over-topping of an uncompleted structure. Therefore, although

studies had shown that a flood like that of 1913 had an average periodicity of about 150 years, the program at each place was so arranged that a flood even as large as that could be handled during construction, either by passing down the original waterway, or through artificial channels, or by partial storage in the basins.

Generally speaking, large floods were most likely to occur during the months of March and April, although some large floods had occurred in the late fall. The summer months were assumed to be reasonably free from extremely large floods, and so in figuring on river closures, or any other step in the construction program which meant a critical situation temporarily, the summer season was chosen as being reasonably free from flood menace. At Germantown, because of the steep watershed, floods were liable to occur more often, and more suddenly, than at the other dams. A five to ten-foot rise within a few hours, was not uncommon.

The first step was to build the conduits, and in order to get capacity enough to handle floods during construction (together with what storage might be available in the basin) it was necessary to build them of greater depth than would be required ultimately, with the idea of filling up the extra depth after the embankment was entirely completed. The valley at the dam site was so narrow that once the river channel was closed, the embankment had to be brought up high enough during one summer season to make it possible to handle a maximum flood by discharge through the conduits combined with storage behind the partly completed dam. That is, there was no room for temporary spillways or temporary diversion channels, except as the conduit design could be modified to serve in the latter capacity.

There was some room in the valley floor, between the river channel and the conduit location, where some embankment could be placed to advantage while the outlet structure was being built. And later, as soon as the barrels of the conduits were finished, this could be extended across them to the hill side, while the rest of the concrete work was being completed. This gave an opportunity to use the suitable material from conduit excavation, cut-off trench, etc., without double handling, and permitted a section of embankment to be started on that side of the river channel, which helped that much when it came to closing off the old channel permanently. It also gave an opportunity to get the

hydraulic fill equipment working smoothly, and to get the work organized, before the critical work of river closure began.

The construction program called for getting camps built, equipment installed, power lines constructed, etc., during the spring of 1918, and to continue as follows:

For season of 1918.

(a)	Build outlet works	
	Earth excavation	100,000 cu. yds.
	Rock excavation	45,000 " "
	Concrete	16,000 " "
(b)	Backfill for same	10,000 " "
(c)	Soil stripping	9,000 " "
(d)	Cut-off trench	7,000 " "
(e)	Steel sheet piling	60 tons
(f)	Embankment north side of river channel	
	Hydraulic or semi-hydraulic fill	55,000 cu. yds.
	Rolled or semi-hydraulic fill	65,000 " "
	Miscellaneous fill	8,000 " "

For season of 1919.

(a)	Soil stripping	11,000 cu. yds.
(b)	Cut-off trench	9,000 " "
(c)	Steel sheet piling	50 tons
(d)	Make river closure and complete embankment to safe height for floods.	
	Hydraulic fill	370,000 cu. yds.
	Rolled or semi-hydraulic fill	65,000 " "

For season of 1920.

(a)	Complete embankment	
	Hydraulic fill	300,000 cu. yds.
(b)	Build spillway	
	Earth excavation	25,000 " "
	Rock excavation	8,000 " "
	Concrete	800 " "
(c)	Fill conduits and place permanent floor.	
	Concrete	1,600 " "
(d)	Start spillway bridge.	

For season of 1921.

Finish, dress and seed slopes of dam.
Build roadway across top with guard rails, etc.
Finish spillway bridge.
Clean up and finish all work by September.

Excavation Plant. The principal excavation items at Germantown consisted of the excavation for the outlet structure, the excavation from borrow pits for concrete aggregates and for embankment material, and the excavation for the spillway. The conduit excavation was partly in earth and partly in rock, the structure being so located that it was all founded on rock, and the conduits themselves were laid in a trench cut through the rock. This rock is what is known as Cincinnatian formation, and consists of alternate layers of shale and limestone. At Germantown the shale predominated, and the limestone layers were comparatively thin and far apart. The bottom of the conduit excavation was below water level, and it had to be protected from flooding by cofferdamming and pumping. The greatest width of cut (for the stilling pool) was about 100 feet. The width of the trench for the conduits themselves was about 30 feet, at the bottom.

The borrow pits for concrete aggregates and for embankment material were located in the river valley above the dam site. To get the quantity required they would extend a half mile or so upstream. The material consisted of river sand and gravel with an overburden of clay and loam. To secure the most suitable material, much of it had to be excavated from below water level.

The excavation for the spillway was comparatively simple, as it required only the opening up of a cut through a saddle in the hills. It could be done economically by a small steam shovel and team outfit, or by any excavating equipment that might be selected for the outlet structure and, therefore was not a real factor in the selection of excavating equipment for the job as a whole.

In choosing the excavating equipment for this job therefore, the conduits and the borrow pits for embankment material were the two controlling factors. For the conduit excavation the choice was between dragline machine and steam shovel. The dragline machine was selected for the following reasons: It could work from the top of the cut, and therefore do the work without danger of interruptions from flooding; it could excavate the conduit trench in the rock to the actual width required, since it could load the material at the higher level, whereas a steam shovel would be obliged to take out extra width to make room for cars and track at grade; it could complete the excavation for the stilling pool, and for the inlet, to full width in one cut, thus per-

mitting concrete work to start as soon as the excavation had gotten well under way, whereas a steam shovel would have to take out several cuts, to get the required width, and therefore would have to practically complete all the excavation before concrete work could begin.

For the borrow pit excavation the choice was between dragline machine (loading into cars), dipper dredge (loading into cars or scows, or pump scows), or suction dredge. Steam shovel work was out of the question here because of the fact that so much of the material had to come from below water level. The dragline machine was chosen because of the many advantages it seemed to have in this particular case. Lower first cost and higher resale value, long reach and flexibility of operation, were some most clearly evident. The transportation problem was easier to work out for a dragline machine than for a dipper dredge. A suction dredge, of course, could pump direct from borrow pit to embankment, but the pipe line would be so long that two or more booster pumps would be necessary, the friction loss for this material running as high as 8 to 14 feet per 100. Wear and tear on the long pipe line would be a serious matter also, although the high carbon dredge pipe which was developed on the job later, would have reduced the cost of pipe replacement to a considerable extent. The output of a dredge pump is much less, pumping blindly from a borrow pit, than with a positive feed out in the open, which is possible when material is brought to the sump in cars. The dragline-and-cars combination made it possible to keep the dredge pumps loaded more nearly to capacity, which meant better progress and lower costs. Preliminary estimates indicated that the dragline and cars would put the material to the site of the dam cheaper than any other plant that was considered. In actual operation this plan was fully satisfactory.

A machine with 80 to 100-foot boom and 2½ to 3½-yd. bucket would have sufficient reach and capacity for both the outlet and borrow pit excavations. A small machine was required to furnish concrete stock and for miscellaneous work which the larger machine would not handle economically. For hauling the waste from the outlet excavation and for transporting the material from borrow pit to hog-box, 12-yd. standard gage dump cars, and 40-ton standard gage saddle tank locomotives were selected. Four-yard cars were considered, but the larger cars are easier to load with a dragline machine as large as the one required for

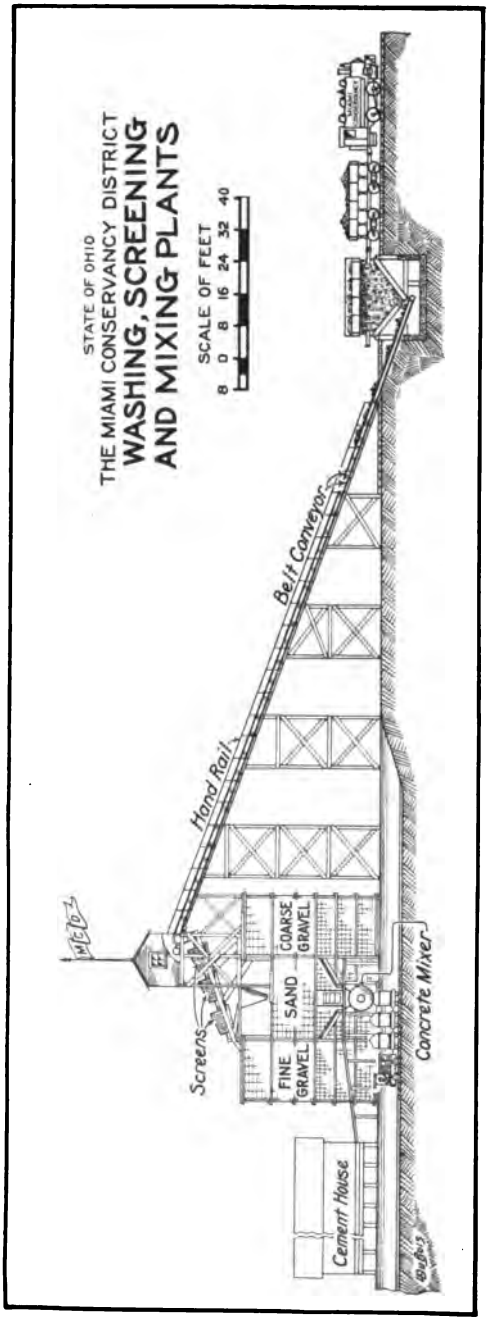


FIG. 25—TYPICAL CONCRETE PLANT AT DAMS

The material is brought to the plant in dump cars and is carried on a conveyor belt up to the screens where the wash water is applied. Storage bins for three sizes of aggregate are provided, from which the material is fed by gravity through adjustable measuring boxes with a charging hopper above the mixer. The mixed concrete is dumped into dump cars or dump buckets which are hauled to the work by means of a narrow gage gasoline locomotive.

this work, and the output is easier handled with the larger equipment. Air compressors and drills were required for drilling the rock in the conduit excavation.

Concrete Plant. Gravel for concrete was available at the dam site by excavation from the river bed. The material had to be washed and screened, requiring a washing and screening plant with enough capacity to permit the work to go ahead on the progress schedule required. For mixing the concrete a 1-yd. mixer was selected as the right size to maintain the required progress. The washing, screening and mixing plant was built as a combined structure of the general type mentioned in Chapter I. Fig. 25 shows the general layout of this plant as used at all of the dams. The material is brought to the plant in dump cars and is dumped into a hopper at the foot of the conveyor belt. A grizzly above the hopper rejects stones and boulders larger than about 6 inches. At the top of the belt conveyor, the material is delivered to the screens, where an ample quantity of wash water removes the dirt and impurities. Material is separated into four sizes: oversize, rejected by 3-in. screens, may either be wasted or may be diverted into the coarse gravel bin, depending on whether the work going on will permit the use of oversize. Except for what oversize may be diverted into it, the coarse gravel bin contains the material between 1½ and 3-in. The fine gravel bin contains the ⅜-in. to 1½-in. material. The sand bin takes the material smaller than ⅜-in. All this material passes direct from screens to bins, by gravity. The concrete mixer is set up in a position such that it may be fed by gravity from each of the bins, and that the mixed concrete may be poured direct into dump buckets or dump cars as shown. Instead of placing a floor in the bins at the elevation of the discharge chutes, the bottoms of the bins were simply allowed to fill up with dead material. This saves the expense of a floor and also serves to give stability to the whole structure. The capacity of the bins was about one-half day's supply of concrete material. Adjustable measuring boxes built into the chutes between bins and mixer allowed the proportions of the various sizes of material to be changed at will.

Cement was stored in a cement house adjacent to the plant, and the cement was taken to the mixing platform as needed, by means of small warehouse trucks. The cement house in the illustration shows the floor at an elevation convenient for unloading

direct from cars. At Germantown, where all cement was hauled in by truck, the cement house was placed in the slope of the side-hill, with the floor far enough below the road so that all the cement could be chuted from the truck into place in the house by gravity. Except for such slight modifications as these, these plants were the same at each of the dams. Each had a capacity of about 350 cu. yds. per 10-hour day.

The plant at Germantown was erected at a convenient point just upstream from the conduit location. The small dragline machine listed under excavation equipment was used to excavate the gravel from the borrow pit, and the material was hauled to the screening plant in 12-yd. dump cars. The mixed concrete was transported from the mixing plant to the job in 1¼-yd. side hopper dump cars, hauled by 3-ton gasoline locomotives. The average haul was approximately 750 feet. The track was high enough so that most of the concrete could be chuted into place by gravity direct from the cars. Where the structure was so wide that the chutes would not reach all the way on a suitable grade, concrete buggies were used to put the material where it belonged.

A small portable concrete mixer was required also for odds and ends of work, either disconnected from main job, or which for some reason had to be put in at a later date.

In the list of equipment which is made part of this chapter, only the equipment itself connected with the concrete plant is included. The bins, cement house, etc., at Germantown cost about $5,000 and the the cost of dismantling was several hundred more. This is a good illustration of the distinction between plant and equipment, the equipment in this case being less than one-half the cost of the plant as a whole. This also prompts the suggestion that in estimating plant cost for a job, a sufficient amount must be added to equipment prices to cover freight, installation, additional construction like storage bins, housing facilities, etc., and for dismantling and cleaning up when the job is finished.

Hydraulic Plant. As soon as the preliminary investigations at the dam sites were finished, and before the selection of equipment began, it was decided that all five of the dams would be built by the hydraulic fill method. As has been said before, the most suitable material for embankment purposes for this hydraulic fill at Germantown, lay in the river valley above the dam site. Having determined upon the dragline excavator with loco-

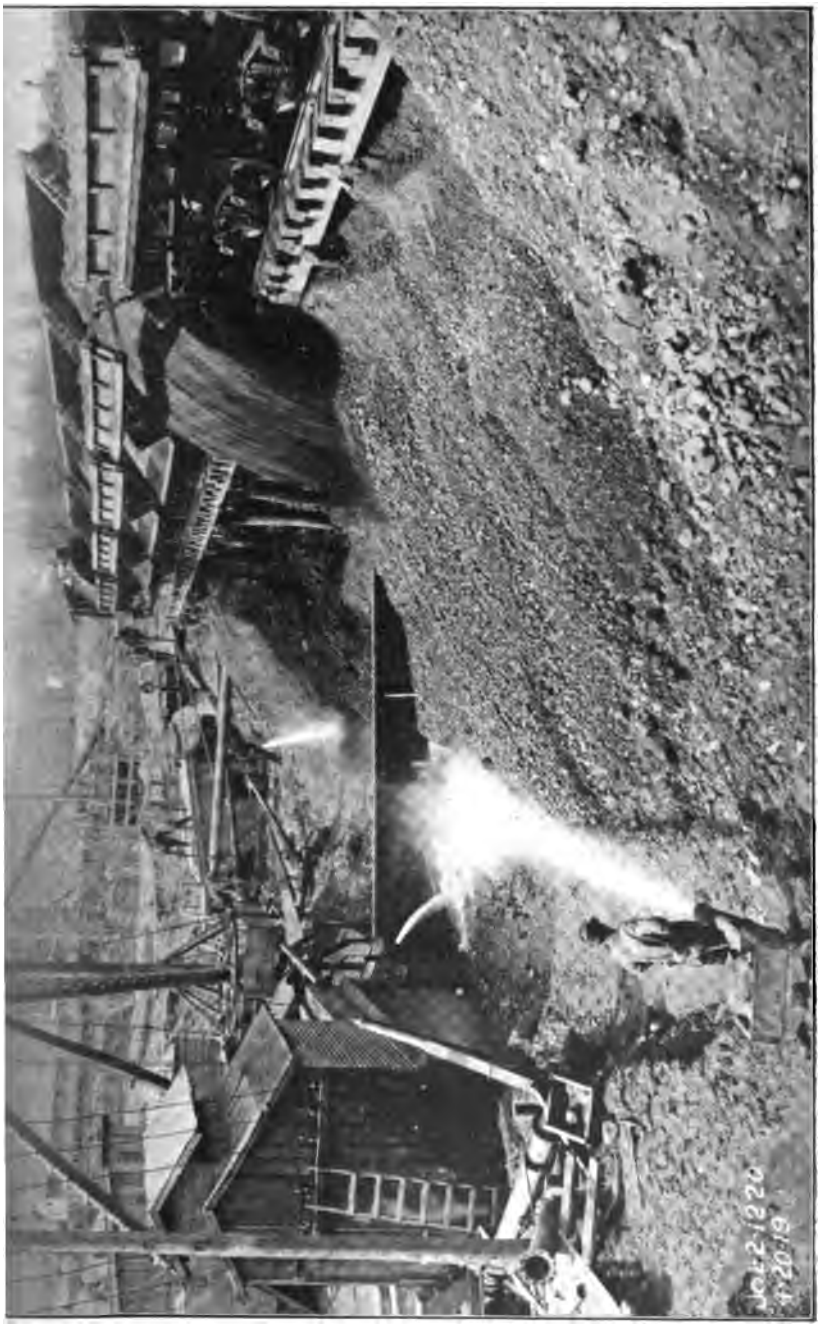


FIG. 26—HOG-BOX STRUCTURE AT ENGLEWOOD DAM

Material from the borrow pit is hauled to the hog-box in 12-yd. dump cars. Hydraulic giants under low pressure wash the material from the pile to the dredge pump sumps.

tives and cars for excavating equipment, there were required the necessary dredge pumps, pipe, etc., to put this material from the hog-boxes into the dam embankment. The "hog-box" is the structure comprising the dumping platform, the troughs leading to the dredge pump sumps, and the sumps themselves. Fig. 26 is a picture of one of them at the Englewood Dam. The one at Germantown was similar. Hydraulic giants were used to sluice the material from the pile where the cars left it, into the sumps where the pumps picked it up. These were No. 2 giants equipped with 2½-in. and 3-in. nozzles and were operated at about 60-lb. pressure. The dredge pumps were located in the building at the left of the picture, and the entrance to the sumps is in the corner of the hog-box next the building. There are two dredge pumps, with separate sumps, and the material may be fed into either sump by working on the material at one end or the other of the hog-box. High pressure pumps were needed to supply water to the giants, but the giants did not furnish enough water to keep the dredge pumps going, so an additional supply of water was needed for that purpose. This was obtained at Germantown by pumping from the river, and low pressure pumps were required for that purpose.

The borrow pits contained some rocks and boulders too large to be handled by the dredge pumps. These were rejected by a revolving grizzly between the dumping platform and the sump. A derrick was used to take away this oversize, and dump it into a pile out of the way of other operations. A glance at Fig. 26 will show again the difference between equipment and plant. The whole hog-box outfit as illustrated in this picture is a part of the construction plant on the job. The equipment, consisting of giants, pumps, pipe, etc., in this case, as in the case of the concrete plant, is only a small part of the plant construction. It comprises only that part of the plant which has a transfer value. Obviously, most of this hog-box is of value only where it stands and for the purpose for which it was built.

After this work had started at Germantown, it developed that there were not sufficient fines in the borrow pit material to keep up the required volume of impervious core in the hydraulic fill. Therefore a borrow pit for clay was opened up on the hillside above the hog-box, and this clay was torn down from the bank by means of hydraulic giants of the same type as those used at the hog-box, but they were operated under high pressure.

More pressure pumps and pipe were required for that work. This material was sluiced direct into the hog-box where it joined the material brought in by the cars.

Power Distribution Plant. On account of the extensive use of electric power, transformers and transmission line equipment was required for distribution on the job. As has been mentioned before, the electric power was brought to the job by the local company, and was taken over by the job as "high tension energy." The preliminary estimate of power requirements for the Germantown job was 4,400,000 K.W.-hrs., with a maximum demand of 900 K.W.

Shop Equipment. As has been mentioned in Chapter I, a small shop was installed at each of the important jobs, to take care of repairs and other work which could be handled better on the job than at the central machine shop. These local shops were also equipped to handle the necessary work on form building, etc. The shop equipment at Germantown was similar to that at the other dams and included band, rip and cross-cut saws, ordinary blacksmith equipment, a chain hoist suitable for handling heavy parts, and other miscellaneous equipment, the need for which will be apparent upon reference to the list.

Miscellaneous Plant. The equipment such as derricks, hoists, pile drivers, unwatering pumps, etc., which cannot well be allocated under the other headings, is included in the list under "Miscellaneous Plant." No comment is required regarding the equipment so listed at Germantown.

Equipment List. The following list shows the equipment in use at the Germantown Dam listed under various headings. At the end of the list is given a figure representing a total plant cost at Germantown which, as has been explained, is considerably in excess of the total equipment cost, inasmuch as it includes also freight charges, installation cost, including bins, trestles, hog-boxes and similar structures, and an estimated charge for dismantling, all of which are charged against the plant account, and must be distributed to cost of items of work.

Table XVII.—Equipment List—Germantown Dam

Note: Items purchased second hand are marked S.H. All others were new.

Excavating Equipment

		Price each
1	Lidgerwood Class K electric dragline excavator, skid mounted, 100-ft. boom, 2½ to 3½-yd. bucket, main hoist engine driven by 112-H.P. General Electric slip ring motor. Swing engine operated by 52-H.P. General Electric motor, both motors no load speed, 720 R.P.M. 30 minute rating. Total working weight of machine 110 tons. S.H.	$ 20,833.00
1	Marion Model 36 dragline excavator, with 40-ft. boom, 1½-yd. bucket, 8″ x 9″ main hoist engine, 6″ x 7″ swing engine, boiler 60″ x 9-ft. 6-in., rated 42-H.P. Total weight of machine mounted on crawling traction trucks, 51 tons	15,100.00
18	Western air dump cars, 12-yd. standard gage, 19-ft. wood body, 50,000-lb. capacity trucks, complete standard M.C.B. equipment.	
	10 cars @	1,625.00
	8 cars @	1,930.00
2	American 40-ton saddle tank locomotives, type 040, standard gage, cylinders 14″ x 22″, complete M.C.B. and Westinghouse air equipment	13,000.00
7000	feet 60-lb. and 70-lb. rail; 1200 feet 30-lb. rail	3,280.00
1	Sullivan air compressor, Class W.G. 3, size 10″ x 10″, single stage, 213-cu. ft. displacement at 100 lbs., 235 R.P.M. Belted to 40-H.P. Allis-Chalmers motor with speed of 1150 R.P.M. Vertical air receiver, 30″ x 72″	1,438.00
4	Sullivan type 2 F.L. 3, steam tripod rock drills, 3¼″ cylinder	325.00
2	Ingersoll-Rand butterfly jackhammer drills, style 100 B.C.R. 4, S.H.	40.00
1	Lambert hoisting engine and boiler; double drum, double cylinder 10″ diameter x 12″ stroke. Two winch heads, 11″ x 11″. S.H.	1,500.00
1	Erie City 50-H.P. locomotive type boiler, mounted on trucks. S.H.	800.00
1	Caldwell No. 104 standard water tank; 14-ft. diameter, 8-ft. 5-in. deep; made of Louisiana cypress	220.00

Concreting Equipment

1	Dull gravel washing and screening plant. 3 revolving screens, 54″ x 27″ x 72″ with ⅜″, 1½″ and 3″ holes, 60″ conical sand separator, 5-ply ½″ x 20″ x 350′ conveyor belt, all operated by 20-H.P. Wagner motor. Also 3½″ American Well Works centrifugal pump, direct connected to 25-H.P. Wagner motor. No bins or other structures included	2,319.00

		Price each
1	Smith concrete mixer, No. 28-S, 1-cu. yd. capacity, mounted on wood skids, power tilt, batch feeding hopper, gear driven by 20-H.P. Allis-Chalmers motor	1,880.00
1	Smith concrete mixer, size 7-S, ¼-cu. yd. capacity, power side loader, mounted on trucks, with 5-H.P. Smith horizontal gas engine	776.00
6	Concrete cars, 1¼-cu. yd. capacity, radial gate, side hopper dump, 18″ wheels, 36″ gage	119.00
5	Sanford-Day flat cars, 36″ gage, 18″ Whitney roller bearing wheels, 3″ oak bed, 55″ square	202.00
2	Plymouth gasoline locomotives, Model A.L. 3-ton, 36″ gage	1,725.00
5	Insley concrete buggies, all steel, 6-cu. ft. capacity, S.H.	19.00
1	Huber tank wagon; 450-gal. capacity. S.H.	225.00

Hydraulic Equipment

2	United Iron Works 15″ dredge pumps; bottom discharge, 36″ runner, 86′ head, 6,500 G.P.M. @ 514 R.P.M., direct connected on common base to 350-H.P. Allis-Chalmers motor, type ANY variable speed 350 to 514 R.P.M.	5,445.00
4	Hydraulic giants. Size No. 2, ball bearing head, 2½″ and 3″ nozzles, made by Joshua Hendy Iron Works	206.00
2	Allis-Chalmers 8″ centrifugal pumps. Type S, single stage, 1950 G.P.M., 140′ head, 1750 R.P.M., bottom discharge. Direct connected to 100-H.P. Allis-Chalmers motor	1,400.00
1	Allis-Chalmers 14″ centrifugal pump. Type S, single stage, 6600 G.P.M., 46′ head at 690 R.P.M., direct connected to 100-H.P. Allis-Chalmers motor	2,460.00
143	feet Continental 18″ wood stave pipe, 100′ head	310.00
2080	feet American Rolling Mills 15″ electrically welded, special analysis, slip joint dredge pipe with fittings	6,000.00
	Wrought iron pipe, ranging from 4″ to 15″	2,000.00
590	feet American spiral riveted pipe with fittings. Asphalt coated, 10″ and 15″	2,990.00
1	Revolving grizzly; 4-ft. diameter, 12-ft. long, with 7″ perforations; driven by 7½-H.P. motor	1,425.00
1	Bedford steel guy derrick, 10-tons capacity, 85-ft. boom, 95-ft. mast, 16-ft bull wheel, mounted on 10″ x 10″ skids; operated by a Lidgerwood double drum electric hoist, friction type, with attached swinger. General Electric motor, 37-H.P. 600 R.P.M. S.H.	5,500.00

Power Distribution Equipment

2	General Electric transformers—400KVA, 33000/2300 volts, outdoor type H. Oil cooled	1,767.00

		Price each
1	General Electric transformer—400 KVA, 29700/33000/2300 volts. Outdoor type, oil cooled	1,876.00
1	General Electric galvanized steel tower. Type T-1617417, for transformer station. 15′ x 9′ base, 20′ high, galvanized iron shapes	210.00
1	General Electric lightning arrester. Catalogue No. 196481, 27000/37900 volts; sphere gaps DL-1500806; cone filler; set of copper tubing; 16-gal. K.D. ammonia solution; type I, form C-4-DO	1,225.00
1	General Electric disconnecting sub-station switch, type L, form G19. 35000 volts, 300 amps	285.00
2	Wagner transformers, type H.E. 10 KVA, 2300/230/115 volts, model 53-N	107.00
2	Wagner transformers, 100 KVA, 2300/460/230 volts, model 85-N, type H.E	519.00

Note: All motors 3-phase, 60-cycle.
All transformers single phase, 60-cycle.

Shop Equipment

1	Famous 36″ band saw, No. 303, with tilting table and belt drive. Made by Sidney Tool Co	273.00
1	Oshkosh No. 1 rip and cross-cut table saw, mounted	319.00
1	Superior 20″ drill press; square base, wheel and lever feed, stationary head	141.00
1	Blount No. 3 emery grinder, floor type. Suitable for 2 emery wheels	47.00
1	Rivet forge, 24″, with No. 4 Buffalo blower	25.00
1	Buffalo blower, No. 4	25.00
1	Hay Budden anvil, 195-lb	37.00
1	Yale & Towne 2-ton triplex chain hoist; 9-ft. lift, 11-ft. reach, 120-lb. max. pull	91.00
1	10-H.P. Allis-Chalmers motor, 440 volts, 1150 R.P.M., type "AN"	272.50
1	American Model "A" gasoline storage outfit. One-gal. measuring pump, 282-gal. tank	148.00

Miscellaneous Equipment

1	Wooden stiff leg derrick. Made in M.C.D. shops. 24-ft. mast, 32-ft. boom, 32-ft. legs, mounted on 27-ft. sills. Operated by Dobbie single drum hand winch	450.00
1	American hoisting engine and boiler, 20-H.P., double drum, double cylinder, 5½″ x 8″, with winch heads. S.H.	320.00
2	Vulcan No. 2 steam pile drivers; cylinder 10½″, stroke 36″, 3000-lb. striking weight, 50-ft. leads of 8″ x 10″ pine.	
	1 S.H.	575.00
	1 New	700.00

		Price each
2	Weinman centrifugal pumps, class E, 3″ suction, 2″ discharge, 80-ft. head at 1750 R.P.M., 35-ft. at 1150 R.P.M. Direct connected to 3-H.P. Allis-Chalmers motor	218.00
1	Swaby centrifugal pump, 10″ suction, 8″ discharge; direct connected to vertical steam engine. S.H.	350.00
1	Gould "Pyramid" pump, 6 x 12, capacity 114 G.P.M., max. head 175-ft. Belted to 7½-H.P. General electric motor with automatic float switch control	422.00
1	Caldwell standard water tank, No. 117, 16-ft. diameter by 13-ft. 5-in. deep, with float and conical cover; made of Louisiana cypress	427.00
1	Gem City 20-H.P. horizontal boiler, on skids	609.00

The total cost of the equipment items for the Germantown work was $162,000. These lists include equipment only, that is, those articles which have a value and use when considered apart from the plant as a whole, and which may be transferred from plant to plant, or used independently.

In contrast with this, the cost of *plant* at Germantown, including the above equipment together with foundations, temporary railroads, shop and warehouse buildings, material for bins, and other non-transferable items that go to make up a working construction plant, was $265,400.00

To this must be added the cost of installation (setting up equipment, erecting bins, building hog-boxes, etc.), mostly labor charges 57,600.00

And the cost of dismantling 17,500.00

Making the total plant investment at Germantown.. $340,500.00

All of which, however, was not necessarily on the job at one time, nor for the full construction period.

The total cost of the Germantown job was $1,340,000, of which the total plant investment was 25.4%. Of this total plant investment the value of equipment items only, was 48%.

Englewood Dam

Description. The Englewood Dam is located on the Stillwater River, about 10 miles above the City of Dayton. The valley at this point is nearly a mile wide and a high dam was required in order to provide the necessary capacity in the basin. These conditions resulted in the Englewood Dam being the largest, as well

as highest, of the five dams constructed in connection with the flood control project.

The dam is an earth embankment with a pair of concrete conduits of the same type as those at Germantown. (See Fig. 23.) The total volume of the embankment was about 3,600,000 cu. yds. of which about 3,400,000 cu. yds. were placed by the hydraulic fill method. The rest of the material, which was used for blanketing the upstream portion of the foundation, building cross-dams to confine various sections of the work, and for other miscellaneous purposes, was placed either as puddled fill or as rolled fill. The dam has a maximum height of 124 feet. It measures 4,750 feet along its crest, which is 30 feet wide and carries a roadway across the valley. Both the upstream and downstream slopes vary from 2 to 1 at the top to 4 to 1 at the bottom with 10-foot berms, the first berm 20 feet below the top and the others 30 feet apart. (See Fig. 22.)

The outlet conduits are located just west of the river channel. They are horse-shoe shape, as at Germantown, and of similar cross-section, about 13 feet wide and about 10½ feet high. For stream control purposes they also were first built about 14 feet deeper, and afterwards filled in as was done at the Germantown dam. These conduits at Englewood are 709 feet long from head-wall to head-wall with flaring wingwalls at the inlet end and a standard stilling pool, similar to the one at Germantown, at the outlet end. The total amount of concrete in the outlet works is 17,850 cu. yds. The spillway is located at the west end of the dam and consists of an ogee overflow weir 100 feet long with its crest 16 feet lower than the top of the dam. This spillway is spanned by a reinforced concrete girder bridge which carries the roadway across the spillway gap, and also forms an overhead crossing over an interurban electric line. The total amount of concrete in spillway and bridge is 8,600 cu. yds.

Railroad Facilities. The nearest railroad to this work was the Dayton, Toledo & Chicago R. R., which passed up the Stillwater Valley about one-half mile west of the dam site. A right of way was secured, leading from this railroad over to the work, and a connection was built in to the job, and was run to various points on the work as necessity required. This made it possible to ship in to this job direct by railroad, all carload shipments of equipment, materials and supplies, which greatly facilitated construction operations. Materials and supplies which could be handled

more economically by shipping in carload lots to the central warehouse at Dayton, and distributed to the jobs in smaller quantities, were delivered to Englewood by trucks. A main highway and an interurban traction line from Dayton crossed one end of the dam site. Fig. 41 (Chapter IV) shows the general layout of this job.

Construction Program. The stream control problem at Englewood was much more troublesome than at Germantown, because the Englewood Dam was so large and such large quantities were therefore required in the lower portion of it, that it was not possible, in one season, to bring it up to a height sufficient to turn a maximum flood through the enlarged conduits. This meant that the dam had to be built in sections, and even with that plan it was not feasible to make the river closure all in one season, without the use of a temporary spillway at an elevation somewhat higher than the river bed.

The first step in construction was to build the conduits, and for the purpose of stream control it was necessary to build them of greater depth than would be required ultimately, and to fill up the extra depth after the embankment was completed, as was done at Germantown, and for the same reasons. At the same time a section of the hydraulic fill embankment could be built on the side of the river channel opposite the conduits, with a cross dam along the river bank to retain the hydraulic fill section. This cross dam was built as a puddled or rolled fill, and the material in it was carefully selected to correspond to the distribution of coarse and fine material in the cross section of the dam itself.

The next season it was expected to close off the river channel and to bring the fill up to a height sufficient to turn a maximum flood through the temporary spillway, the bottom of which was some 35 feet higher than the old river channel. The next season the temporary spillway gap was to be closed and the whole dam could then be brought up to sufficient height to turn a maximum flood through the enlarged conduits. Even by thus making this river closure in two steps, large quantities of embankment were involved and sufficient equipment had to be obtained to keep up the scheduled rate of progress without question.

The construction program called for the building of camps, installation of equipment, power service, etc., during the spring of 1918, and was to be continued as follows:

For season of 1918.

(a)	Build outlet works.	
	Earth excavation	11,000 cu. yds.
	Rock excavation	43,000 " "
	Concrete	20,000 " "
(b)	Backfill for same	28,000 " "
(c)	Soil stripping	33,000 " "
(d)	Cut-off trench	11,000 " "
(e)	Trim up river channel.	
(f)	Embankment, east side of river channel.	
	Rolled or puddled fill	325,000 " "
	Hydraulic fill	410,000 " "
(g)	Temporary spillway.	
	Earth excavation	45,000 " "

For season of 1919.

(a)	Soil stripping	21,000 cu. yds.
(b)	Cut-off trench	6,000 " "
(c)	Steel sheet piling	67.9 tons
(d)	Complete temporary spillway.	
(e)	Make river closure, bringing embankment high enough to turn floods through temporary spillway.	
	Rolled or puddled embankment	187,000 cu. yds.
	Hydraulic fill	532,000 " "
(f)	Excavation for permanent spillway.	
	Earth excavation	110,000 " "
	Rock excavation	7,000 " "

For season of 1920.

(a)	Soil stripping	19,000 cu. yds.
(b)	Cut-off trench	11,000 " "
(c)	Close temporary spillway gap, and bring the completed embankment to safe height for floods.	
	Rolled fill	18,000 " "
	Hydraulic fill	960,000 " "
(d)	Spillway concrete	6,000 " "

For season of 1921.

(a)	Complete embankment.	
	Hydraulic fill	1,200,000 cu. yds.
	Complete spillway and spillway bridge.	

Concrete ..... 2,600 cu yds.

Fill conduits and place permanent floor.

Concrete ..... 2,200 " "

For season of 1922.

(a) Finish, dress and seed slopes of dam.

Build roadway across top with guard rails, etc.

Clean up and finish all work by September.

This was the construction program as originally laid out and on the basis of which the equipment was selected. It was changed somewhat later, because a more favorable location was found for the temporary spillway, but the resulting changes in the program had no material effect on the requirements for equipment. Less embankment than is specified here was placed during the first season, but these quantities were exceeded in later periods, so that all embankment was completed during the year 1921.

Excavation Plant. As at Germantown, the principal excavation items at Englewood were for outlet conduits, for concrete aggregate, for embankment materials, and for the spillway. The outlet structure was founded on rock and the conditions were similar to those described in connection with the Germantown work, except that the Englewood rock had a larger percentage of limestone layers, and the shale was not quite as hard as at Germantown. The excavation for the conduits themselves consisted of a trench cut to neat lines through the rock, the bottom being below water level. The bottom of the stilling pool was still deeper, so that cofferdamming and pumping were necessary to keep this work from flooding. The greatest width of cut was about 120 feet, and the conduit trench was about 30 feet wide.

Borrow pit conditions at Englewood were similar to those at Germantown except that they were much more extensive. They extended about a mile upstream from the dam, and were more than a third of a mile wide in places. The material was similar to that at Germantown, and much of it had to be taken from below water level.

The excavation for the spillway was a side hill cut in Cincinnatian rock. Except for one or two springs which emerged from

the hillside at this point, this spillway was a dry job. Therefore, as was the case at Germantown, the excavation for conduits and for the borrow pits were the two controlling factors in the choice of excavating equipment.

The selection of plant for conduit excavation presented the same problem as at Germantown and a dragline machine was selected for excavating for the same reasons. For the borrow pit excavation the conditions were practically the same as at Germantown, except that the quantities were much larger at Englewood and for that reason several dragline machines, and more cars and locomotives were required. Because of the greater length of the borrow pit at Englewood, in considering the possibility of using a suction dredge at this site the disadvantage was even greater than at Germantown, on account of the greater length of pipe line that would be required.

Concrete Plant. Gravel for concrete was available near the dam site at a borrow pit above the elevation of ground water. One of the dragline machines listed under excavation equipment was used to excavate this gravel, which was hauled to the screening plant in 12-yd. dump cars. A washing, screening and mixing plant duplicating the one at Germantown was selected for this work also, and was erected near the upstream end of the conduits. The mixed concrete was transported from the mixing plant to the job in 1½-yd. hopper side dump cars, hauled by 3-ton gasoline locomotives. The average haul was about 600 feet. Most of the concrete in the floors and walls of the conduits was chuted into place, but it was not feasible to elevate the track enough to so handle all of it, so a traveling crane and buckets were used for the tops of the conduits, a derrick was required to reach the far side of the stilling pool, and a tower and chutes were used for a portion of the inlet walls.

The concrete for the spillway was put in at a later date and was too far from the main concrete plant for the transportation of mixed concrete. What was done there was to take the washed and screened material from the main plant, transport it to the spillway in standard 12-yd. cars, and there feed it to a mixing plant which was set up at the spillway job for that purpose. This mixing plant consisted of a 1-yd. mixer, with suitable charging hoppers, measuring boxes, etc. This mixer discharged into 1-yd. bottom dump buckets which were handled by a 100-foot guy derrick.

Hydraulic Plant. The plan for getting the material from the borrow pits into the embankment was the same at Englewood as at Germantown, except that because of the greater length of the Englewood Dam, a somewhat different arrangement of hog-boxes was necessary.

In the first place, in order to maintain a safe control of the river during construction, it was necessary to build the Englewood Dam in sections, retaining the original waterway in the river channel, or providing an artificial one, until such time as the shore sections of the dam had been brought up to an elevation sufficient to turn a large flood through the conduits.

For the first section of the dam a hog-box was located near enough to the river channel, and at such elevation, that the water supply was brought to it by gravity. Because of the high friction loss in the discharge lines from the dredge pumps (10 to 14 feet per 100), it was found to be more economical to use other hog-box locations for the other sections of the dam, than to try to build it from one. In order to avoid long pipe lines and excessive discharge head, three other hog-boxes were used in addition to the first one before the dam was completed. These were located along the upstream slope of the dam at convenient points with respect to the distribution of yardage in the dam, and high enough in elevation so that the material could be put into place in the embankment without the aid of booster pumps. It was found cheaper to elevate the material by pulling the trains up a grade than to elevate it in the discharge lines by means of additional pumps. During a part of the time, two or three of these hog-boxes were in use one after another, as the fill progressed from one end of the dam to the other, this process being repeated as the embankment was raised, one lift at a time. When three of them were so used, the dredge pump motors were moved from No. 1 hog-box to No. 3, and back again, while No. 2 (the middle one) was in commission, to save the expense of buying extra 500-H.P. motors.

In listing hydraulic equipment the original installation of dredge pumps is listed. Replacements of worn-out parts, shells, runners, etc., were classed as supplies, and charged directly to the item of work. On a job the size of Englewood Dam therefore a large number of these parts were used up, which do not appear on the equipment list.

To emphasize again the difference between *plant* and *equipment,* one of these hog-boxes at Englewood, exclusive of equipment, cost between $10,000 and $15,000. All four of them were figured in as a part of the plant cost, but in the equipment list only a part of that cost appears.

Other Requirements. The requirements for power distribution at Englewood were similar to those at Germantown, except that they were much larger. It was estimated that a peak load of 2,000 K.W. must be provided for, and that a total of about 18,000,000 K.W.-hrs. would be used.

The lists of shop equipment and miscellaneous plant speak for themselves.

Table XVIII.—Equipment List—Englewood Dam

Note: Items purchased second hand are marked S.H. All others were new.

Excavating Equipment

	Price each
2 Bucyrus Class 24, electric dragline excavators, skid mounted, 85-ft. boom, 4½-yd. bucket or 100-ft. boom, 3½-yd. bucket, main hoist engine driven by 200-H.P. type M Westinghouse motor; swing engine operated by 75-H.P. type M Westinghouse motor. Total working weight, 120 tons	$ 38,650.00
1 Bucyrus Class 24, steam dragline excavator, skid mounted, 100-ft. boom, 3½-yd. bucket, main engine 10½" x 12" double cylinder; swing engine 8" x 8" double cylinder; boiler 58" x 15-ft.; working weight 120 tons	34,675.00
1 Lidgerwood Class B steam dragline excavator, truck mounted, 60-ft. boom, 1½-yd. bucket; main engine 9" x 10" double cylinder, double drum, 35-H.P.; swing engine 6¼" x 8" double cylinder, single drum, 12-H.P.; boiler 60" x 9'-6" rated @ 59-H.P.; total working weight of machine 50 tons. S.H.	13,500.00
2 Marion Model 36 dragline excavators, with 40-ft. boom, 1½-yd. bucket, 8" x 9" main hoist engines, 6" x 7" swing engine, boiler 60" x 9'-6", rated at 42-H.P. Total weight of machine mounted on crawling traction 50 tons.	
1 @	15,100.00
1 with extra steam shovel boom @	17,500.00
43 Western air dump cars, 12-yd. capacity, standard gage, side dump, 19-ft. wood body, 50,000-lb. capacity trucks, complete standard M. C. B. equipment.	
32 cars @	1,625.00
11 " @	1,930.00

		Price each
7	American 40-ton saddle tank locomotives, type 040, standard gage, cylinders 14" x 22", complete M.C.B. and Westinghouse air equipment.	
	6 @	13,000.00
	1 @	11,200.00
44850	feet 56-lb. rail and 1128 feet 30-lb. rail	26,180.00
1	Sullivan air compressor, Class W.G. 3, size 10" x 10", single stage, 213-cu. ft. displacement at 100-lb., 250 R.P.M. Belted to a 40-H.P. Allis Chalmers motor with speed of 1150 R.P.M. Vertical air receiver 30" x 72"	1,438.00
1	Ingersoll-Rand wagon drill, "A" frame type, standard 20-ft. turntable, G-64 Dake hoisting and traction engine, 35-H.P. boiler; machine mounted on four flanged wheels. S.H.	1,500.00
3	Sullivan jack hammer drills, class D.P. 33, solid piston Rotator type	155.00
1	Flat car, 50-ft. long, standard gage. S.H.	500.00
1	Caldwell No. 104 standard water tank, 14-ft. diameter, 8'-5" deep, made of Louisiana cypress	220.00
1	Erie City 50-H.P. locomotive type boiler mounted on truck S.H.	800.00
	Concreting Equipment	
1	Dull gravel washing and screening plant, 3 revolving inclined screens, 54" x 27" x 72", with ⅜", ½" and 3" holes, 60" conical sand separator, 5-ply ½" x 20" x 350' conveyor belt, all operated by 20-H.P. Wagner motor. Also 3" American Well Works centrifugal pump, direct connected to 25-H.P. Wagner motor. No bins or other structures included	2,319.00
1	Smith concrete mixer, No. 28-S, 1-cu. yd. capacity, mounted on wood skids, power tilt, batch feeding hopper, gear driven by 20-H.P. Allis-Chalmers motor	1,880.00
1	Smith concrete mixer, size 7-S, capacity ¼-cu. yd., power side loader, mounted on trucks, with 5-H.P. Smith horizontal gas engine	776.00
6	Concrete cars, 1¼-yd. capacity, radial gate, side hopper dump, 36" gage	119.00
2	Lockewood concrete buckets, 1½-yd., double bail, bottom dump. S.H.	100.00
2	Plymouth gasoline locomotives, model A.L., 3-ton, 36" gage	1,725.00
	Hydraulic Equipment	
2	Morris, 15" dredge pumps, 135-ft. head, horizontal bottom discharge, with base and direct connection to 500-H.P. Allis-Chalmers 2200-volt motors, variable speed 350 to 514 R.P.M.	8,904.00
2	American Manganese 15" dredge pumps, 135-ft. head, horizontal bottom discharge, direct connected to	

		Price each
	500-H.P. Allis-Chalmers 2200-volt motors, variable speed 350 to 514 R.P.M.	7,344.00
2	American Manganese 15" dredge pumps, 135-ft. head, horizontal bottom discharge, direct connected to 350-H.P. Allis Chalmers 2200-volt motor, variable speed 350 to 514 R.P.M.	6,295.00
6	Hydraulic monitors, No. 2 base, ball bearing head, 2½" and 3" nozzles, made by Joshua Hendy Iron Works	206.00
1	Allis-Chalmers 12" centrifugal pump, type S, double suction, single stage, 80-ft. head, 1170 R.P.M., 5500 G.P.M., direct connected to 150-H.P. Allis-Chalmers motor	2,040.00
2	Cameron 10" centrifugal pumps, type D.V. single stage, double suction, 140-ft. head (casing for 280' head in series), 1740 R.P.M., 1950 G.P.M., direct connected to 100-H.P. Westinghouse motor	1,989.00
1	Morris 8 stage pump, 325-ft. head, 1700 R.P.M., 100-125 G.P.M., driven by 25-H.P. Wagner motor. S.H.	765.00
2848	feet American Rolling Mills 15" electrically welded, special analysis slip joint dredge pipe, with fittings	4,571.00
2560	feet Thomas 15" slip joint, riveted dredge pipe	7,296.00
6	Revolving grizzlys, 4-ft. diameter, 12-ft. long, with 7" perforations, driven by 7½-H.P. motor	1,425.00
1	All steel guy derrick, 8 tons capacity, 85-ft. boom, 95-ft. mast, 16-ft. bull wheel; mounted on 10-ft. x 10-ft. skids; operated by Lidgerwood double drum electric hoist, friction type, with attached swinger, 37-H.P. General Electric motor, 600 R.P.M. S.H.	5,500.00
	Power Distribution Equipment	
4	Packard transformers—400 KVA, 33000/2300 volts, type "A"	1,517.00
3	General Electric transformers—400 KVA, 33000/2300 volts, type "H," form VDD	1,757.00
1	General Electric galvanized steel tower, for transformer station, type T, 1617417, 15-ft. x 9-ft. base, 20-ft. high, galvanized iron shapes	210.00
1	General Electric lightning arrester, catalogue 196481, 27000/37900 volts; sphere gap DL1500806; cone filler; set of copper tubing; 16-gal. K.D. ammonia solution; type I, form C-4-D.O.	1,225.00
1	General Electric disconnecting sub-station switch, 35000 volts, 300 amps, type L, form G19	285.00
3	Wagner transformers—100 KVA, 2300/460/230 volts, type HE, model 85-N	519.00
6	Wagner transformers—75 KVA, 2300/460/230 volts, type HE, model 85	431.00
2	Wagner transformers—10 KVA, 2200/220/110 volts, type HE, model 53M	106.00

Note: All motors 3-phase, 60-cycle.
All transformers single phase, 60-cycle.

Shop Equipment

	Price each
1 Famous 36" band saw, Sidney Tool Co., No. 303, with tilting table	$ 273.00
1 Oshkosh No. 1 rip and cross-cut table saw, mounted	319.00
1 Superior 20" drill press, square base, wheel and lever feed, stationary head	141.00
1 Blount No. 6 emery grinder, floor type, suitable for two emery wheels	47.00
1 Royal No. 40 steel forge, 10" fan, 22-ft. hearth, 30" high	27.00
1 Buffalo No. 3 steel pressure blower	36.00
1 Hay-Budden anvil, 195-lb	37.00
1 Yale & Towne 5-ton "Triplex" chain hoist	196.00
1 Yale & Towne 2-ton "Triplex" chain hoist	91.00
1 10-H.P. Allis-Chalmers motor, 440 volts, 1150 R.P.H., type "AN"	272.50
1 American model "A" gasoline storage outfit with measuring pump, 282-gal. tank	148.00

Miscellaneous Equipment

1 Orton Steinbrenner 20-ton locomotive crane, steam operated, 45-ft. boom, 1-yd. clamshell bucket, 8 wheels, maximum reach 47-ft.; lifting capacity, 57,000 lbs. at 10-ft radius and 5,200 lbs. at 60-ft. radius	21,000.00
1 All steel guy derrick, 8 tons capacity, 85-ft. boom, 95-ft. mast, 16-ft. bull wheel, mounted on 10-ft x 10-ft. skids; operated by double drum Lidgerwood electric hoist, friction type with attached swinger, 37-H.P. General Electric motor, 600 R.P.M. Coal unloading. S.H	5,500.00
1 Wooden stiff leg derrick—12" x 12" yellow pine, 30-ft. mast, 65-ft. boom, 12-ft. bull wheel	700.00
1 Mundy hoisting engine, 9" x 10" double drum, double cylinder, with 30-H.P. Clyde boiler. S.H	1,500.00
1 Mundy hoisting engine, 8" x 12" double drum, double cylinder, with 30-H.P. boiler. S.H	1,000.00
1 Vulcan steam pile driver, No. 2, 10½" cylinder, 36" stroke, 3000-lb. striking weight, 50-ft. leads, of 8" x 10" pine. S.H	475.00
1 Weinman centrifugal pump—3" suction, 2" discharge, class "E," direct connected to 3-H.P. Allis-Chalmers motor	218.00
1 Gould "Pyramid" pump, 6 x 12, capacity 175 feet head, 114 G.P.M., belted to 7½-H.P. General Electric motor with automatic float switch control	422.00

	Price each
1 Caldwell standard water tank, No. 117, 16-ft. diameter by 13′-5″ deep, with float and conical cover; made of Louisiana cypress	427.00
1 Gem City, 20-H.P. horizontal boiler	609.00

The total cost of the equipment items for the Englewood work was 490,900. This was for equipment only, and included only such articles as had an independent value and use when considered apart from the plant as a whole.

The *plant* cost at Englewood, including the above equipment, together with foundations, material for bins, temporary railroad and other constructions which go to make up a working plant, was	$757,600.00
Adding to this the cost of installation (erecting bins, setting up equipment, assembling machines)	64,600.00
And the cost of dismantling	19,400.00
The total plant investment at Englewood was	$841,600.00

All of which, however, was not necessarily on the job at one time, nor for the full construction period.

The total cost of the Englewood job was $3,600,000, of which the total plant investment was 23.4%. Of this total plant investment the cost of equipment items only, was 58%.

Lockington Dam

Description. The Lockington Dam is located on Loramie Creek, about two miles north of the point where it enters the Miami River, and five miles north of Piqua. This dam is 6,400 feet long, and is the longest of the five flood control dams. Its maximum height is 78 feet, but the deep section of the valley is comparatively short, and a considerable portion of the dam at either end is nothing more than an earth levee 15 feet or less in height. The dam contains about 1,000,000 cu. yds. of embankment of which 795,000 cu. yds. were placed by the hydraulic fill method. About 195,000 cu. yds. were placed by a dragline machine working from a borrow pit alongside, the material being compacted by puddling. The rest of the material came from the cut-off trench or other excavation, and was placed by other methods. The cross section of the embankment is the same, in general, as at Germantown and Englewood, modified as required to fit the topography and the lesser height of dam.

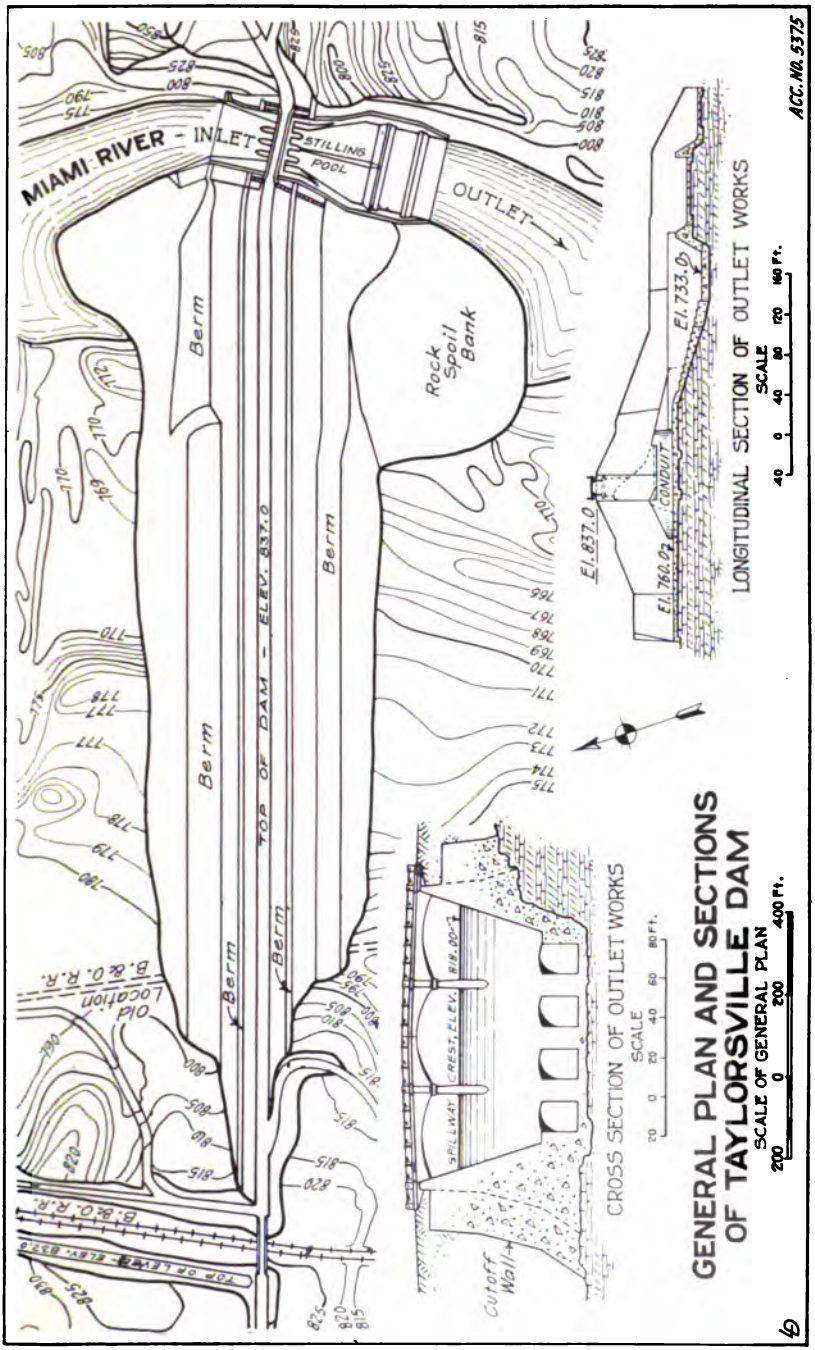


FIG. 27—GENERAL MAP TAYLORSVILLE DAM
Showing general relation between dam and outlet spillway structure.

At Lockington, the outlet conduits and the spillway are combined in a single concrete structure, containing about 32,000 cu. yds. of concrete. This structure was located a short distance east of the river channel and was founded on Springfield limestone. It consists of two retaining walls facing each other, about 27 feet apart at the bottom and flaring towards the top, with a concrete floor between, at river bed elevation, thus forming a gap through the embankment. Across this gap, with its crest 16 feet below the top of the dam, is an ogee overflow weir 62 feet high, forming the spillway, and through the base of this weir are two outlet conduits each 9 feet wide and 9 feet high with vertical sides and an arched roof. At the downstream end is a concrete stilling pool of the same general design as at Germantown and Englewood. A two-span reinforced concrete girder bridge 85 feet long, carries a roadway (along the crest of the dam) over the spillway gap. Fig. 27, which shows the layout of the Taylorsville job, illustrates this type of structure.

Railroad Facilities. There was no railroad near the damsite, but an interurban electric line which passed within about ¼ mile of the damsite made freight connections with a steam railway, so that freight shipments could be delivered to this job by building a short spur in to the work from a convenient point on the electric railway. Main highways passed near the damsite on both sides of the river.

Construction Program. The stream control problem at Lockington was very much simpler than at Germantown or Englewood, due to the different design of the outlet structure. By postponing the construction of the spillway section until after the embankment was completed, the rest of the structure formed a concrete lined gap through the embankment of sufficient size to pass a maximum flood, requiring only a reasonable amount of embankment in place to adequately block off the old channel and turn the water through this gap.

The first step then was to excavate for, and build that part of the outlet structure consisting of the retaining walls, the floor and the concrete stilling pool. Then by completing the diversion channels above and below this structure, the permanent channel was provided for the new course of the stream.

According to the construction program the camp buildings were to be put up, equipment installed, power lines constructed,

etc., during the spring of 1918, and the work was to continue as follows:

For season of 1918.

(a) Build outlet works, except spillway weir and conduits.

Earth excavation	100,000 cu. yds.
Rock excavation	8,000 " "
Concrete	29,000 " "

(b) Soil stripping ..... 10,000 " "

Semi-hydraulic or rolled fill ..... 100,000 " "

(c) Embankment in upstream toe of dam.

For season of 1919.

(a) Complete diversion channels to and from outlet works.

Earth excavation ..... 28,000 cu. yds.

(b) Soil stripping ..... 15,000 " "

(c) Cut-off trench ..... 43,000 " "

(d) Make river closure, and complete embankment to safe height for floods.

Hydraulic fill	590,000 " "
Semi-hydraulic or rolled fill	75,000 " "

For season of 1920.

(a) Complete embankment.

Hydraulic fill	200,000 cu. yds.
Semi-hydraulic or rolled fill	50,000 " "

(b) Build spillway weir and conduits.

Concrete ..... 2,500 " "

For season of 1921.

Finish, dress and seed slopes of dam.

Build spillway bridge.

Concrete ..... 500 cu. yds.

Build roadway across top, with guard rails, etc.

Clean up and finish all work by June.

Excavation Plant. The excavation for the outlet and spillway structure was mostly in gravel which was suitable for use in the concrete, although there was some rock excavation for the foundation. The total excavation for the structure itself amounted to about 42,000 cu. yds., of which 37,000 cu. yds. was gravel. The new channels connecting the structure with the old river

channel required the excavation of another 100,000 cu. yds. of gravel. The structure covered an area approximately 550 feet long and the excavation down to bed rock had a depth of about 18 feet, with an average width of about 100 feet. The excavation in the limestone was only a few feet in depth except at the bottom of the stilling pool, where the greatest depth of cut in the rock was about 17 feet.

For the excavation for this structure a dragline machine was selected, because, with its long reach, it could set the stripping off to one side and pass a large portion of the conduit excavation over within easy reach of the screening and washing plant for concrete aggregates, thus cleaning up the whole job in one or two cuts. For finishing up the rock excavation (and it could also be used for placing the concrete) a guy derrick was chosen, as it could place some of this rock as plums in the concrete or could load it into cars for use as slope paving. Dump wagons and dump carts, loaded by hand, were also used in disposing of the rock excavation.

At Lockington the most suitable material for hydraulic fill lay in the slope east of the river channel upstream from the dam and at an elevation somewhat higher than the valley floor. This made it possible to sluice the material down to a point near the toe of the dam where the dredge pumps were to be located. Here, therefore, was a somewhat different problem than that presented at either Germantown or Englewood, and the natural decision was to use hydraulic giants to tear down the banks in the borrow pit, and additional water under low pressure to sluice this material down to the sumps. Dragline and cars, and steam shovel and cars, were also considered for this work, but it was estimated that sluicing would be considerably cheaper than any other method that could be used.

For a section of the embankment west of the deeper valley section a dragline machine seemed to be the most suitable equipment, because there was suitable material alongside the site of the embankment which could be excavated by a dragline machine and placed direct into the fill, only requiring such treatment by puddling, or other sufficient methods, as to give it the necessary compactness. The same machine which handled the excavation for the outlet structure would be available for this work, as this section of embankment would not be built until after the outlet excavation had been completed.

Concrete Plant. The excavation for the outlet and spillway structure furnished all the gravel necessary for concrete aggregates. One of the standard washing, screening and mixing plants duplicating the ones at Germantown and Englewood was set up for this work. A small dragline machine loaded material out of the storage pile, into 4-yd. dump cars, which were hauled by a narrow gage steam locomotive to the receiving hopper of the screening plant. While the excavation was still in progress and before the small dragline arrived, the big machine loaded these cars, incidentally, in connection with its regular work of excavation. Later, for placing the concrete in the spillway section and the bridge, the small dragline machine opened up a pit close to the plant and fed the material direct to the receiving hopper. The mixed concrete was transported from the mixing plant to the job in 1½-cu. yd. bottom dump buckets on narrow gage flat cars hauled by a gasoline locomotive. Two guy derricks with 129-foot mast and 105-foot boom were used to handle these buckets at the job. These derricks were used also for handling the form panels for the walls.

Hydraulic Plant. As the material at Lockington for the hydraulic fill portion of the dam was all sluiced direct to the dredge pumps, more elaborate hydraulic equipment was required than at either Germantown or Englewood. Higher pressures were required at the giants in the borrow pits, in order to tear down the material and break it up into condition for sluicing. This required larger giants and more high pressure pumps than were required at those dams where the giants were used only at the hog-boxes. In order that operations might be continuous, branch lines of high pressure pipe were used so that the operations could be continued in one part of the pit, while the giants were being moved up at another place.

The sluicing water at Lockington was obtained from the Miami and Erie canal, and was brought to the high pressure pumps by gravity. The low pressure water was carried on to the borrow pits by gravity, and was distributed to the various points at the top of the cut by a system of ditches.

On account of the relatively small amount of hydraulic fill at Lockington and the more favorable conditions for river closure, it was felt that 14-in. dredge pumps with 250-H.P. motors and 12-in. discharge lines, would be large enough. Two of these pumps were required for the first installation and they were set

up so that they could be used either in parallel or in tandem. As work progressed it was expected that additional pumps would be required because of longer lines and high elevations. A third pump with a 350-H.P. motor was installed in such a way that one of the three could be used as a booster for either of the other two. As a matter of fact, however, a booster was not required. The larger motor and long shoes on the runner were sufficient to put the material out to the extreme points. Eventually, as the original pump shells required replacement, 15-in. pumps were substituted, so as to make the pumping equipment interchangeable on all of the jobs, but the 12-in. discharge line at Lockington was used throughout the job.

As the borrow pit became worked out, flat grades for the ground sluice resulted, and the ground water in the sump prevented lowering the grade at that end. To meet this situation one of the dredge pumps was used to pump down the ground water, and the discharge from this pump was carried up to the upper end of the borrow pit, thus providing additional sluicing water, as well as permitting steeper sluicing grades. About the same time the small dragline machine was moved into the borrow pit and used to excavate some of the material too low to be sluiced, piling it up where it could be under-cut and washed down by the ground sluice.

For excavating the sump, setting the pumps, and manipulating the suction pipes, trash screens, etc., a stiff-leg derrick was erected at a commanding point.

Power Distribution Plant. For a service track into the job a spur was built in from the Western Ohio Electric Railway line, with a trolley line connected with the company's overhead system. Power for general use on the job was obtained from the Dayton Power & Light Company's plant at Piqua, over a 33,000 volt transmission line carried up to the dam especially for that purpose. The Piqua power plant was afterwards connected up with the company's central station, making this line more dependable, as a part of the general system. For distribution over the work the necessary equipment of transformers, lightning arresters, and low voltage transmission lines was required. The estimated power requirements were about 4,000,000 K.W.-hrs. with a peak load of 1,000 K.W.

Other Requirements. The shop equipment required at Lockington was practically the same as at Germantown and Engle-

wood, and no special comment is required as to miscellaneous plant except as regards the grouting machine.

It has been mentioned before that the foundation of the outlet structure was on Springfield limestone. This rock is seamy and it was anticipated that grouting of the foundation might be necessary, in order to prevent seepage under and around the structure at times of back water in the basin. As soon as possible after work started test pits were opened up in the rock to determine the presence of seams, if any, and the result of these investigations indicated that grouting would be desirable. A grouting machine was therefore secured for this purpose as listed under miscellaneous equipment. The rock drills and compressor listed under excavation equipment were also used in this connection.

Table XIX.—Equipment List—Lockington Dam

Excavating Equipment

Note: Items purchased second hand are marked S.H. All others are new.

		Price each
1	Lidgerwood Class K electric dragline excavator, truck mounted, 100-ft. boom, 2½-yd. bucket, main hoist engine driven by 112-H.P. General Electric slip ring motor. Swing engine operated by 52-H.P. General Electric motor, both motors no load speed 720 R.P.M., 30-minute rating. Total working weight of machine, 110 ton. S.H.	$ 20,833.00
1	Lidgerwood Class B steam dragline excavator, truck mounted, 60-ft. boom, 1½-yd. bucket. Main engine 9″ x 10″ double cylinder 35-H.P. Swinging engine, 6¼″ x 8″ double cylinder, 12-H.P. Boiler, 60″ x 9′-6″, rated at 50-H.P. Total working weight of machine 50 ton. S.H.	9,000.00
8	Western dump cars, 4-yd. wood body, 36″ gage, hand dump. S.H.	94.00
1	Vulcan steam locomotive, class CS, 040, 36″ gage, saddle tank, cylinders 9″ x 14″, working weight 15-ton, tractive force 5301-lb. S.H.	1,700.00
5000	feet 30-lb. rail, 5310 feet 60-lb. rail, 6 30-lb turn-outs, 2 60-lb. turn-outs	5,168.00
1	Sullivan air compressor, type W.G. 3, size 10″ x 10″. Single stage, 213-cu. ft. displacement at 100-lbs. and 250 R.P.M., belted to a 40-H.P. Allis-Chalmers motor. Speed 1150 R.P.M. With 30″ x 72″ vertical air receiver	1,438.00
1	Ingersoll-Rand tripod rock drill, type E44, air or steam operated with E-42 gland	330.00

		Price each
1	Ingersoll-Rand butterfly jack hammer drill, style 100 BC4, rotator type	137.00
1	Sullivan jap drill, class DP 33, solid piston rotator	137.00
1	Little Western 2-horse road grader, 6-ft. 14-in. blade	125.00
3	Dump wagons. S.H.	
	1 @	100.00
	2 @	135.00
2	Kilbourne-Jacobs contractor's dump carts	77.00
1	Erie City 50-H.P. locomotive type boiler mounted on trucks. S.H.	800.00
	Concreting Equipment	
1	Dull gravel washing and screening plant, 3 revolving screens, 54" x 27" x 72" with ⅜", 1½" and 3" holes, 60" conical sand separator, 5-ply ½" x 20" x 350-ft. conveyor belt. All operated by 20-H.P. Wagner motor. American Well Works centrifugal pump direct connected to 25-H.P. Wagner motor. No bins or other structures included	2,319.00
1	Smith concrete mixer, No. 28S, 1-cu-yd. capacity, on wood skids, power tilt, and batch feeding hopper, gear driven by 20-H.P. Allis-Chalmers motor	1,880.00
1	Smith concrete mixer, size 7-S, power side loader, on trucks, with 5-H.P. Smith horizontal gasoline engine	776.00
7	Clark concrete buckets, 1½-yd. capacity, double bail, bottom dump. S.H.	100.00
5	Sanford-Day flat cars; 36" gage, Whitney roller bearing wheels	202.00
2	Plymouth gasoline locomotives, model A.L. 3-ton, 36" gage	1,725.00
2	Bedford, all steel, 8-ton guy derricks, 120-ft. mast, 105-ft. boom, 14-ft. bull wheel, operated by a Lidgerwood electric hoist, double drum friction type, with 52-H.P. General Electric motor, and auto brake swinging gear	5,568.00
1	Caldwell standard water tank, No. 104; 14-ft. diameter, 8'-5" deep; made of Louisiana cypress	220.00
	Hydraulic Equipment	
2	United Iron 14" special dredge pumps, 45" runner, bottom discharge, direct connected to 250-H.P. 220-volt Allis-Chalmers motor, variable speed, 350 to 514 R.P.M.	5,810.00
1	American Manganese 15" dredge pump, reducing suction and discharge elbows to 12" pipe, 115-ft head, 4200 G.P.M., direct connected to 350-H.P. Allis-Chalmers motor, variable speed 350 to 514 R.P.M.	6,180.00
1	Hydraulic Giant, size No. 4, made in M.C. D. shops, 2½" and 5" nozzles	496.00
3	Hydraulic Giants, size No. 2, ball bearing head, 2½" and 3" nozzles, made by Joshua Hendy Iron Works	206.00

		Price each
2	Allis-Chalmers 8" centrifugal pumps, type S, double suction, single stage, 140-ft. head, 1750 R.P.M., 1950 G.P.M. bottom discharge, direct connected to 100-H.P. Allis-Chalmers motor	1,400.00
1	Allis-Chalmers 6" centrifugal pump, type S, double suction, single stage, 150-ft. head, 1000 G.P.M., direct connected to 60-H.P. Allis-Chalmers motor	1,100.00
1	Allis-Chalmers 14" centrifugal pump, type S, double suction, single stage, 90-ft. head, 1175 R.P.M., 6500 G.P.M., direct connected to 200-H.P. Allis-Chalmers motor	2,765.00
2000	feet American Rolling Mills, 12" slip joint, special analysis, dredge pipe with fittings	5,605.00
2000	feet American 15" spiral riveted pipe with fittings	7,200.00
1460	feet wrought iron pipe, ranging from 4" to 10", with fittings	1,090.00
1	Derrick, made in M. C. D. shops, 60-ft. steel boom, 30-ft. wood mast, wood stiff leg, operated by a Lidgerwood electric hoist, double drum, friction type, with 52-H.P. General Electric motor	5,065.00
	Power Distribution Equipment	
1	General Electric transformer, 400 KVA, 33000/2300 volts, type H. form V.D.D.	1,767.00
3	Packard transformers, 400 KVA, 33000/2300 volts, type A	1,517.00
1	General Electric galvanized steel tower for transformer station, type T, 1617417, 15′ x 9′ base, 20′ high, galvanized iron shapes	210.00
1	General Electric lightning arrester; catalogue No. 196481, 27000/37900 volts, sphere gap DL1500806; cone filler; set of copper tubing; 16-gal. K.D. ammonia solution; type I, form C-4-D.O.	1,225.00
1	General Electric disconnecting sub-station switch, 35000 volts, 300 amps., type L, form G19	285.00
3	Wagner transformers, 100 KVA, 2300/460/230 volts, type HE	519.00
	Note: All motors 3-phase, 60-cycle. All transformers single phase, 60-cycle.	
	Shop Equipment	
1	Famous 36" band saw, Sidney Tool Co., No. 303, complete with tilting table, belt drive	273.00
1	Oshkosh, No. 1 rip and cross-cut table saw	319.00
1	Superior 20" drill press, square base, wheel and lever feed, stationary head	141.00
1	Blount No. 6 emery grinder, floor type, suitable for two emery wheels	47.00
1	Royal No. 40 steel forge, 10" fan, 22" hearth, 30" high	27.00
1	Hay-Budden anvil, 195-lb	37.00
1	Champion No. 2 blower	10.00

		Price each
1	Champion blower, 12" fan	10.00
1	Cyclone 3-ton chain hoist	119.00
1	Cyclone 1-ton chain hoist. S.H	45.00
1	Allis-Chalmers 10-H.P. motor, 440 volts, 1150 R.P.M., type AN	272.50
1	American model A gasoline storage outfit, with measuring pumps, 282-gal tank	148.00
	Miscellaneous Equipment	
1	Ransome pressure grouting machine, ½-cu. yd. capacity, barrel type, operated by air. S.H	200.00
1	Mundy hoisting engine and boiler, 8" x 12" double drum, double cylinder, with winch heads. S.H	1,000.00
1	National swing engine, single drum, double cylinder, 5" x 8" horizontal reversing type. S.H	500.00
1	Platt 14" centrifugal pump, double suction, single stage, 30-ft. head, 865 R.P.M., 1000 G.P.M., direct connected to 50-H.P. Allis-Chalmers motor. S.H	974.00
1	Gould "Pyramid" pump, 6 x 12, capacity 175-ft. head, 114 G.P.M., belted to 7½-H.P. General Electric motor, with automatic float switch control	422.00
1	Worthington centrifugal pump, class C, type 6, 8" suction, 6" discharge, belted to a 30-H.P. Allis-Chalmers motor, 1150 R.P.M	498.00
1	Worthington centrifugal pump, class C, size 4, 5" suction, 4" discharge, belted to a 20-H.P. Allis Chalmers motor, 1150 R.P.M	388.00
2	Weinman centrifugal pumps, class E, 3" suction, 2" discharge, direct connected to 3-H.P. Allis-Chalmers motor	218.00
1	Domestic 4" diaphragm pump connected to 1½-H.P. Domestic gas engine, mounted on 4-wheel truck	220.00
1	Caldwell standard water tank, No. 117, 16-ft. diameter by 13'-5" deep, with float and conical cover, made of Louisiana cypress	427.00

The total cost of these equipment items, for the Lockington work was $123,100. This covered equipment only, that is, only such articles as were transferable from job to job and had an independent value apart from the plant as a whole.

The *plant* cost at Lockington, including the above equipment together with foundations, bins, temporary railroads and other non-transferable parts of the working plant, was $205,600.00

Add the cost of installation (erecting bins, assembling machines, setting up equipment, etc.) 61,800.00

And the cost of dismantling 4,600.00

Making the total plant investment at Lockington $272,000.00

All of which, however, was not necessarily on the job at one time, nor for the full construction period.

The total cost of the Lockington job was $1,280,000 of which the total plant investment was 21.3%. Of this total plant investment, the cost of equipment items only, was 45%.

Taylorsville Dam

Description. The Taylorsville Dam is on the Miami River about ten miles above Dayton. The river valley at the site of the dam is about 3000 feet wide from hill to hill, and to take advantage of the available storage capacity in the basin, a dam with a maximum height of about 78 feet was required.

The dam is an earth embankment of the same general cross-section as the others that have been described, with a combination outlet and spillway structure such as was used at Lockington. The total volume of the embankment was 1,290,000 cu. yds., of which about 910,000 cu. yds were placed by the hydraulic fill method. Two cross dams were required, during the course of construction, to cut off sections of the hydraulic fill, as was done at Englewood. The material in these cross dams was placed as rolled fill, or was compacted by puddling. That class of work amounted to about 215,000 cu. yds. in all. A large quantity of random fill amounting to approximately 165,000 cu. yds. was placed around one end of the outlet structure and across the old river channel. For this, waste rock from the outlet excavation was used.

The outlet structure was the largest single concrete structure built by the District. It contains 48,000 cu. yds. of concrete, and made necessary the excavation of more than 700,000 cu. yds. of earth and rock. The retaining walls on either side have a maximum height of nearly 90 feet; the crest of the spillway is 132 feet long; the conduits are four in number, each 19 feet high and 15 feet wide. The spillway gap is spanned by a three-span reinforced concrete girder bridge of the cantilever type. Another bridge spans the gap at the west end of the dam where the relocated Baltimore & Ohio Railway passes up the valley. This is a single-span reinforced concrete, three hinged arch, and contains about 500 cu. yds. The highway passing along the crest of the dam and crossing these two bridges has now become a part of the National Road, one of the main thoroughfares for east

and west traffic across the country. Fig. 27 shows the layout of this job.

Railroad Facilities. A section of the Baltimore & Ohio Railroad running from Cincinnati to Toledo passed through this dam site at an elevation not far above the valley floor. This railroad was relocated so as to take it around the edge of the retarding basin. It furnished adequate railroad facilities for the job while construction was in progress, first from the original railroad line, and later from the relocated line after it was put into service, a switch and siding being installed at the dam site for this purpose. Branch lines to various parts of the job were run out from the main siding. There was no main highway within a mile or more of this damsite, but it was served by fairly good secondary roads of dirt and gravel. Trucking to this job was attended with some difficulty.

Construction Program. The type of outlet structure at Taylorsville, being similar to the one at Lockington, made the stream control problem comparatively simple. After the walls and floor of the structure were completed the river could be diverted through this new channel and a waterway was thus made available for handling large floods. At Taylorsville, however, the volume of water to be handled during a flood was very large, much larger than at any of the dams described so far, and in order to confine the maximum flood flow within the limits provided, a large volume of embankment in the dam must first be placed.

The excavation quantities for the outlet structure were very large, and a considerable portion of this excavation was suitable for use in the dam. In order to thus make use of it therefore, arrangements were made to put it into a section of the embankment west of the river channel where there was sufficient room available to take care of it. This also helped along the matter of river closure, as it lessened the quantities necessary to bring the embankment up to the required height at the time the river closure was made. Meanwhile the relocation of the Baltimore & Ohio Railroad was to be made so as to interfere as little as possible with embankment construction on that side of the river.

Another thing that affected stream control, and the closure of the river section, was that during the 1913 flood a deep hole had been scoured in the river bed at this point, which afterwards

had been filled up with mud and silt. Before the river channel section of the dam could be placed, this hole had to be coffer-dammed off and pumped out, so as to get rid of the mud and silt and provide a suitable foundation for the embankment. This meant that a considerable period of time must elapse from the time the river was diverted through its new channel until the embankment which was to effect permanent closure could be started. This made it all the more imperative that once the river channel section was started, the work progress as rapidly as possible until the required height was reached, because all this work had to be done during a low water period. The river at Taylorsville was not subject to the flashy floods that might be expected at Germantown, but otherwise the flood probabilities were similar, and river closure work had to be scheduled for the summer months, when the chance of a big flood was remote.

The first thing to do at Taylorsville, as at the other dams, was to build the outlet structure, or at least that part of it consisting of the retaining walls, the floor and the concrete stilling pool. This was much more of a job than was the similar work at the other dams, on account of the larger quantities of excavation. In disposing of this excavation, as has been mentioned heretofore, the building of a section of embankment was involved, so the two were started simultaneously.

During the spring of 1918 the construction camp was to be erected, equipment was to be installed, power lines constructed, etc., so that actual work could begin as soon as possible. Two construction trestles were required across the river channel, one to carry a standard gage track and a wagon roadway, and the other to carry the discharge pipe from the dredge pumps. These were put up as a part of the preliminary operations. Work was to continue as follows:

For season of 1918.

(a) Excavate for the foundation of the outlet structure, and start the placing of concrete.

Earth excavation	175,000	cu. yds.
Rock excavation	115,000	" "
Concrete	25,000	" "

(b) Soil stripping ..... 17,000 " "

(c) Cut-off trench ..... 10,000 " "

(d) Build section of embankment, west of river channel, using material from outlet structure excavation.

Hydraulic fill.....130,000 cu. yds.
Semi-hydraulic or rolled fill.....100,000 " "

For season of 1919.

(a) Complete diversion channels to and from outlet works.

Earth excavation.....360,000 cu. yds.
Rock excavation..... 60,000 " "

(b) Complete outlet structure, except spillway and conduits.

Concrete ..... 20,000 " "

(c) Soil stripping..... 5,000 " "

(d) Cut-off trench ..... 5,000 " "

(e) Complete embankment west of river channel.

Hydraulic fill.....340,000 " "
Semi-hydraulic or rolled fill..... 20,000 " "

For season of 1920.

(a) Divert river through new channel.

(b) Cofferdam off and clean up foundation for river channel section.

Excavation ..... 5,000 cu. yds.

(c) Make river closure and complete that section of embankment to safe height for floods.

Hydraulic fill.....420,000 " "
Miscellaneous fill..... 90,000 " "

For season of 1921.

(a) Complete embankment.

Hydraulic fill.....150,000 cu. yds.
Miscellaneous fill..... 75,000 " "

(b) Build spillway and conduits, including spillway bridge south of outlet works.

Concrete ..... 3,000 " "

(c) Build roadway across top with guard rails, etc.

For season of 1922.

Finish, dress and seed slopes of dam.

Complete roadway and guard rails.

Clean up and finish all work by June.

This preliminary program, like those at the other dams, was modified somewhat as the work progressed, but it served to outline the requirements on which the purchase of plant was based.

Excavation Plant. The excavation for the outlet structure at Taylorsville was one of the important features of the job. This structure was located on the east side of the river valley where a steep cliff of Cincinnatian rock rose up practically from the water's edge. The structure was located far enough into the hillside so that it would rest on a rock foundation, with the conduit floors at about the old river level. This, including the inlet and outlet channels, required an excavation of about 710,000 cu. yds., of which 500,000 cu. yds or more was earth suitable for use in the hydraulic fill embankment. The remainder of the excavation, which was in Cincinnatian rock, contained a large percentage of hard clay and shale, and was not considered suitable for use in the embankment of the dam, because the shale would slack on exposure and therefore would not be desirable even in the toes of the fill. It had to be put somewhere, however, and was used to back up the toes of the embankment on both upstream and downstream sides, as it could be wasted there almost as cheaply as anywhere else.

The topography at Taylorsville was such that a large portion of the earth overburden in the outlet excavation could be sluiced direct to the dredge pumps, and no equipment for handling this material could compete with hydraulic giants under such conditions. For the rock excavation, however, the choice was not so simple. The first thought was for steam shovels and cars, but that involved some serious difficulties. The deepest part of the excavation was to be 40 feet below low water level in the river. The excavation for the structure foundation alone covered an area approximately 600 feet long, and from 150 to 260 feet wide. The pit would have to be protected from flooding by any ordinary seasonal flood, which meant building up dikes and cofferdams to elevations at least 12 to 15 feet higher than ordinary low water, so the bottom of the pit would be 50 to 55 feet below the top of the cofferdams. For steam shovel work under such conditions the tracks would have to be brought out over the top of these protective works, which meant either a long runway out of the

pit, involving additional excavation and much additional protective work for the runways themselves, or exceptionally steep grades to get the tracks out of the pit without making the runways too long. Moreover, the excavation for this structure was 260 feet wide at the downstream end, and many runs would be required with a steam shovel to complete the excavation, and under those conditions none of the concrete work could be started until practically all the excavation had been taken out.

A dragline excavator could take out this material, working from the top of the rock, and loading cars at that elevation instead of in the bottom of the pit. This would do away with the difficult problems of track grades and additional protective works. It also minimized the danger of delays from flooding. A dragline machine could start at one end and complete the excavation to grade and for the full width of the cut as it went, thus permitting the concrete work to begin as soon as the excavation had gotten a reasonably good start. While there was little doubt that a steam shovel would actually load this material into cars faster than a dragline machine could do it, it was believed that taking the work as a whole, the dragline outfit would handle it more cheaply than the other. The final selection therefore, was a dragline machine, truck mounted, with 100-foot boom and 3-yd. bucket. Dump cars of 12-yd. capacity and 40-ton standard gage dinkeys were provided for hauling away the excavated material. A spreader car was used on the spoil banks.

This job required more and heavier drilling equipment than was needed at the other jobs so far described, as reference to the equipment list will indicate.

Suitable gravel for concrete was to be found in the valley floor at several places close to the dam site. Much of the best material lay below the elevation of ground water, and a small dragline machine, with a 60 to 70-foot boom, and 1½ to 2-yd. bucket seemed to be the best suited to handle that excavation. This gravel was hauled to the washing and screening plant in 12-yd. cars by 40-ton dinkeys.

There was considerable work of a preparatory nature, building track grades, benching off for the dragline machine at the outlet structure, and loading some of the overburden that could not readily be handled by sluicing, that was standard steam shovel work. A Model 36 Marion steam shovel-dragline, on caterpillars,

was admirably suited for such work, because it answered the requirements perfectly as a steam shovel, and could be quickly converted into a dragline machine at any time, by simply changing booms. This made the same machine available for use later on the dam, to throw up the dikes along the slopes in connection with the hydraulic fill. Three of these machines were purchased for different jobs, all equipped as draglines, and the one steam shovel boom and dipper was shipped around to whichever machine needed it most at the time.

At the different dams, various methods were used to throw up the dikes along the slopes of the embankment to prevent the discharge from the dredge pipe from overflowing the slope. At some places small dragline machines were used, at other places it was done with teams. During the latter part of the work at Taylorsville an Austin backfiller was obtained for that purpose, and it was by far the most satisfactory and economical equipment for that work.

Concrete Plant. The outlet excavation was cleaned up from the downstream end working upstream. A suitable location for the concrete plant was found just downstream from the site of the outlet structure and there a washing, screening and mixing plant was put up corresponding in all essential details to those described at the other dams. The sand and gravel was obtained from borrow pits on the other side of the river, excavated by the Bucyrus Class 14 dragline machine shown in the list under "Excavating Equipment," loaded into cars, and hauled over to the plant as needed. The mixed concrete was hauled to the job in bottom dump buckets, on narrow gage flat cars, hauled by three-ton gasoline locomotives. These buckets were handled at the job by two 105-foot boom, stiff-leg derricks on skids and rollers, which were moved along upstream as the work progressed. These derricks were also used for handling the forms which were built up into panel sections wherever they could be so used. In crossing the deep part of the stilling pool the derricks and car tracks were supported by substantial crib and trestle work.

Hydraulic Plant. As has been mentioned about 500,000 cu. yds. of the outlet excavation was suitable for use in the embankment of the dam. This material lay high enough in the hillside, so that it could be sluiced direct to dredge pumps. As this mate-

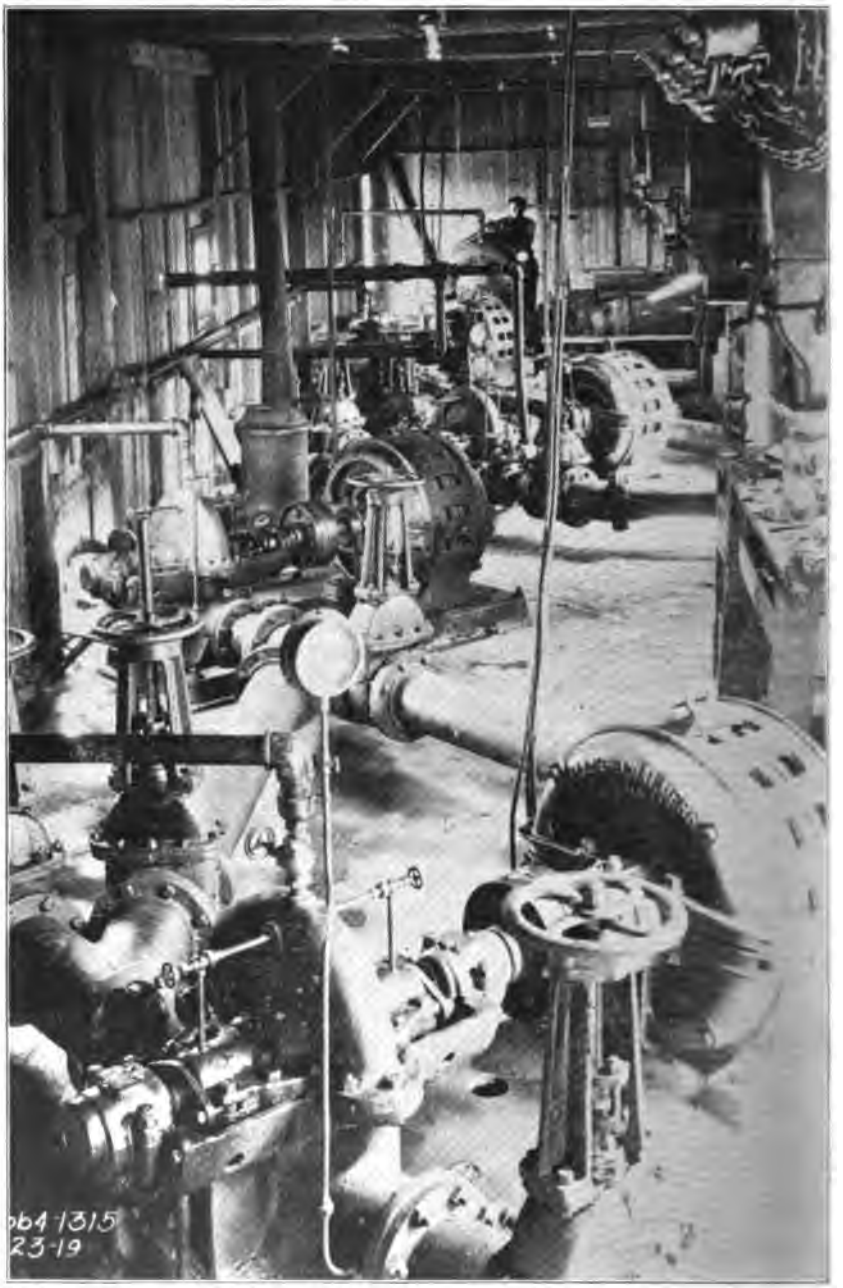


FIG. 28—HIGH AND LOW PRESSURE PUMPS FOR SLUICING OPERATIONS

This shows a part of the pressure pump layout for sluicing operations at Taylorsville Dam.

rial had to be moved from the outlet excavation anyway, the logical place for it was in the dam, and because the sluicing method was cheaper than any other for moving a large portion of this material, hydraulic giants with the necessary equipment of high pressure pumps, and low pressure pumps to supply the sluicing water, were selected for handling that much of the material that was available for the embankment purposes. At first, the question was left open as to whether the additional material for the dam would be secured by extending borrow pits out into the hillside and continuing the same sluicing methods, or whether the rest of the material would be secured from the lower elevation of the river valley by means of dragline and cars as at Germantown and Englewood. The sluicing of the material in the outlet excavation worked so well that the method was continued for all the material required in the dam. The hydraulic equipment at Taylorsville, therefore, was very similar to that used at Lockington, except that 15-in. pipe, instead of 12-in., was used in the discharge line from the dredge pumps. All of the water at Taylorsville for supplying the giants, and for the pressure supply, had to be pumped from the river, requiring more pumping equipment, and of greater capacity, than at Lockington.

Power Distribution Plant. Power for this job was brought in over a special transmission line built for this purpose by the Dayton Power & Light Company. The usual equipment of step-down transformers, lightning arresters, disconnecting switches, etc., was required to distribute this power to the various points on the job. It was estimated that about 4,000,000 K.W.-hrs. would be required with a peak load of 1250 K.W.

Shop Equipment. A shop was erected at Taylorsville similar to those at the other dams and the equipment at this shop was about the same as at the other places, with the addition of a planer and jointer because of the more extensive form work.

Miscellaneous Plant. Under this heading is listed, among other things, a locomotive crane and a railroad ditcher. The locomotive crane was obtained for use in the erection of heavy equipment at this dam, and at other places on the project, and for miscellaneous use on this job and elsewhere.

The American railroad ditcher was purchased originally for use on one of the railroad relocation jobs. It was put into use at Taylorsville on miscellaneous cleaning up work and other jobs for which it is so well suited.

FIG. 29—CONVERTIBLE STEAM SHOVEL-DRAGLINE MACHINE

There were three of these machines on the job equipped as draglines as shown in the upper view. One steam shovel boom and dipper was purchased so that any one of the three machines could be equipped as a steam shovel (illustrated in the lower view) whenever it seemed desirable.

The other items listed under this heading need no comment.

Table XX.—Equipment List—Taylorsville Dam

Note: Items purchased second hand are marked S.H. All others were new.

Excavating Equipment

		Price each
1	Lidgerwood class M electric dragline excavator, truck mounted, 100-ft. boom, 3½-yd. bucket, main hoist engines driven by a 225-H.P. General Electric slip ring motor, swing engines operated by 112-H.P. General Electric motor, both motors no load speed 720 R.P.M. 30-minute rating, total working weight of machine 160 tons. S.H.	$ 20,833.00
1	Bucyrus class 14 steam dragline excavator, skid mounting, 60-ft. boom with 15-ft. extension, 1½-yd. bucket; main hoist engines 8" x 10" double cylinder; swing engines 5" x 6" double cylinder; boiler 54" x 9'-6". Total working weight 75 tons. S.H.	15,500.00
1	Marion model 36 steam shovel-dragline, 8" x 9" main hoist engine, 6" x 7" swing engine, boiler 60" x 113⅜", rated 42-H.P.; dragline equipment 40-ft. boom, 1½-yd. bucket. Total weight of machine mounted on crawling traction trucks 51 tons	15,100.00
	Steam shovel boom and 1½-yd. bucket, extra	2,400.00
1	Austin backfiller, caterpillar mounted, gasoline driven, 30-ft. boom, self loading and dumping scraper. Weight 6 tons	3,150.00
14	Western air dump cars, 12-yd., standard gage, 19-ft. body, 50,000-lb. capacity, complete standard M.C.B. equipment.	
	4 @	1,930.00
	10 @	1,625.00
2	American 40-ton locomotives, type 040, saddle tank, standard gage, cylinders 14" x 22".; complete M.C.B. and Westinghouse air equipment	13,000.00
1	Baldwin locomotive, saddle tank, standard gage, cylinders 15½" x 24", M.C.B. equipment, steam brakes S.H.	6,750.00
20630	feet 60-lb. rail; 750 feet 30-lb. rail	12,306.00
1	Western spreader car; standard gage, main blades 21-ft. x 4'-4" (front end); rear blades 21-ft. x 3-ft.	3,245.00
1	Standard rail laying push car	140.00
2	Sullivan air compressors, class W.G. 3; size 10" x 10", single stage, 213-cu. ft. displacement at 100-lb. and 235 R.P.M., belted to 40-H.P. Allis-Chalmers motor with speed of 1150 R.P.M. With 30" x 6-ft. vertical air receiver	1,438.00
3	Sanderson "Cyclone" class B, non-traction, electric well drills, operated by 7½-H.P. motor	1,615.00

	Price each
4 Ingersoll-Rand tripod rock drills, type E44, air or steam operated, with E42 gland	330.00
1 Ingersoll-Rand butterfly jackhammer drill, style 100 B.C.R. 4. S.H.	40.00
2 Sullivan jap drills; class D.P. 33 solid piston rotator	137.00
2 Kilbourne-Jacobs contractor's dump carts	77.00
1 Caldwell No. 104 standard water tank; 14-ft. diameter, 8'-5" deep, made of Louisiana cypress	220.00
1 Erie City 50-H.P. locomotive type boiler, mounted on trucks. S.H.	800.00
Concreting Equipment	
1 Dull gravel washing and screening plant; 3 revolving inclined screens, 54" x 27" x 72", with ⅜", 1½" and 3" holes, 60" conical sand separator, 5-ply ½" x 20" x 350-ft. conveyor belt; all operated by a 20-H.P. Wagner motor. Also 3½" American Well Works centrifugal pump, direct connected to a 25-H.P. Wagner motor. No bins or other structures included	2,139.00
1 Smith concrete mixer, No. 28-S; 1-cu. yd. capacity, on wood skids, power tilt, and batch feeding hopper, driven by 20-H.P. Allis-Chalmers motor	1,880.00
4 Lakewood concrete buckets; 1½-yd. double bail, bottom dump. S.H.	60.00
5 Clark concrete buckets; 1½-yd. double bail, bottom dump. S.H.	100.00
7 Sanford-Day flat cars; 36" gage, 18" "Whitney" roller bearing wheels	202.00
2 Plymouth gasoline locomotives, model AL; 3-ton, 36" gage	1,725.00
3 Insley concrete buggies; all steel, 6-cu. ft. S.H.	19.00
1 Wallace bar bender; for large size reinforcing steel. S.H.	100.00
2 Bedford all steel 8-ton derricks; 105-ft. boom, 40-ft. mast, 45-ft. stiff leg, 14-ft. bull wheel, operated by a Lidgerwood electric hoist, double drum friction type, with 52-H.P. General Electric motor and auto brake swinging gear	5,568.00
Hydraulic Equipment	
2 Morris 15" semi-steel dredge pumps; 105-ft. head, 505 R.P.M., direct connected to 350-H.P. Allis-Chalmers motor. Variable speed, 350 to 514 R.P.M.	7,345.00
2 American Manganese 15" dredge pumps; 115-ft. head, 4200 G.P.M., direct connected to Allis-Chalmers motor, 350-H.P., 2200 volt, variable speed, 350 to 514 R.P.M.	6,200.00
1 Hydraulic giant, size No. 4, made in M. C. D. shops; 2½" and 5" nozzles	496.00
3 Hydraulic giants, size No. 2, ball bearing head, 2½" and 3" nozzles, made by Joshua Hendy Iron Works	206.00

		Price each
1	Allis-Chalmers 12" centrifugal pump, type S, double suction, single stage, 80-ft. head, 1170 R.P.M., 5500 G.P.M., direct connected to 150-H.P. Allis-Chalmers motor	2,040.00
2	Allis-Chalmers 10" centrifugal pumps, type S, double suction, single stage, 180-ft. head, 1750 R.P.M., 3500 G.P.M., direct connected to 200-H.P. Allis-Chalmers motor	2,200.00
2	Allis Chalmers 8" centrifugal pumps, type S, double suction, single stage, 140-ft. head, 1750 R.P.M., 1950 G.P.M., direct connected to 100-H.P. Allis-Chalmers motor	1,400.00
3	Allis-Chalmers 6" centrifugal pumps, type S, double suction, single stage, 150-ft. head, 1750 R.P.M., 1400 G.P.M., direct connected to 75-H.P. Allis-Chalmers motor	1,165.00
2728	feet American Rolling Mills 15" electrically welded, slip joint, special analysis dredge pipe with fittings	8,187.00
2284	feet Thomas 15" slip joint riveted dredge pipe	6,381.00
730	feet American 15" asphalt coated spiral riveted pipe with fittings	7,140.00
2000	feet Continental 18" wood stave pipe and fittings	3,743.00
646	feet wrought iron pipe, 4" to 15"	750.00
2000	feet 3" black pipe	938.00
1	Revolving grizzly, 4-ft. diameter, 12-ft. long, with 7" perforations, operated by 7½-H.P. motor	1,425.00
	Shop Equipment	
1	Famous 36" band saw, Sidney Tool Co., No. 303, belt driven, complete with tilting table	273.00
1	Oshkosh No. 1 rip and cross-cut table saw	319.00
1	Crescent planer and jointer, 12" head, belt driven. S.H.	160.00
1	Superior 20" drill press, square base, wheel and lever feed, stationary head	141.00
1	Blount emery grinder, floor type, suitable for two emery wheels	47.00
1	Royal No. 40 steel forge, 10" fan, 22" hearth, 30" high	27.00
1	Buffalo No. 3 steel pressure blower	18.00
1	Hay-Budden anvil, 195-lb.	37.00
1	Athol grindstone, 4" face	28.00
1	Yale & Towne 2-ton "Triplex" chain hoist	91.00
1	Allis-Chalmers motor, 10-H.P., 440 volts, 1150 R.P.M., type "AN"	272.50
2	American model A gasoline storage outfits with measuring pump, 282-gal. tank	148.00
	Power Distribution Equipment	
5	General Electric transformers, 400 KVA, 33000/2300 volts, type H, form V.D.D.	1,767.00

	Price each
1 General Electric galvanized steel tower for transformer station, type T, 1617417, 15′ x 9′ base, 20′ high; galvanized iron shapes	210.00
1 General Electric lightning arrester, catalogue No. 196481, 27000/37900 volts, sphere gap DL1500806; cone filler; set of copper tubing; 16-gal. K.D. ammonia solution; type I, form C-4-D.O	1,225.00
1 General Electric disconnecting switch, 35000 volts, 300 amps., type L, form G-19	285.00
3 Packard transformers, 100 KVA 2200/440 volts	533.00
6 Wagner transformers, 100 KVA 2300/460/230 volts, type HE, Model 85N	519.00
3 General Electric transformers, 100 KVA 2300/460/230 volts, type A	622.00
2 Westinghouse transformers 15 KVA 2200/244/122 volts, style 147034	166.00

Note: All motors 3-phase, 60-cycle.
All transformers single phase, 60-cycle.

Miscellaneous Equipment

1 Orton-Steinbrenner 20-ton locomotive crane, steam operated, 45-ft. boom, 1-yd. clamshell bucket, 8 wheels, maximum reach 47 ft., lifting capacity 57,000 lbs. at 10-ft. radius and 5200 lbs. at 60-ft. radius	21,000.00
1 American steam railroad ditcher, $\frac{9}{16}$-yd. bucket, standard upright boiler, equipment to operate clamshell bucket	12,141.00
1 Worthington centrifugal pump, class C, 8″ discharge, 10″ suction, belt driven by 50-H.P. Allis-Chalmers motor	663.00
1 Cameron piston pump, size 7″ x 6″ x 7″, with 8-H.P. Gem City boiler	598.00
3 Weinman centrifugal pumps, 3″ suction, 2″ discharge, class E, 80-ft. head at 1750 R.P.M., 35-ft. head at 1150 R.P.M., direct connected to 3-H.P. Allis-Chalmers motor	218.00
1 Gould "Pyramid" pump, 6 x 12, capacity 175-ft. head, 114 G.P.M., belted to 7½-H.P. General Electric motor with automatic float switch control	422.00
1 Caldwell standard water tank, No. 117, 16-ft. diameter by 13′-5″ deep, with float and conical cover, made of Louisiana cypress	427.00
2 Gem City 20-H.P. horizontal boilers	609.00

The total cost of these equipment items, for the Taylorsville Dam was $286,800. The foregoing lists include equipment only. In other words, only such articles are listed as have a transferable value in themselves and can be used independently.

For comparison with this, the cost of *plant* at Taylorsville, including the equipment listed above, together with foundations, temporary roads, shop and warehouse buildings, and other non-transferable items that go to make up a working construction plant, was $444,300.00

To this must be added cost of installation (setting up equipment, erecting bins, building hog-boxes, etc.), mostly labor items 92,300.00

And the cost of dismantling 23,100.00

Making the total plant investment at Taylorsville.... $559,700.00

All of which, however, was not necessarily on the job at one time, nor for the full construction period.

The total cost of the Taylorsville job was $2,280,000, of which the total plant investment was 24.5%. Of this total plant investment the value of equipment items only, was 48%.

Huffman Dam

Description. The Huffman Dam is located on the Mad River about eight miles east of Dayton, at a point where the valley is about 3600 feet wide from hill to hill. A dam with a maximum height of 73 feet was required at this point.

This dam, like the others, is an earth embankment of the standard cross section, with a combined outlet and spillway structure such as was built at Taylorsville and Lockington. (See Fig. 27). The total volume of the embankment is 1,396,000 cu. yds., most of which was placed by the hydraulic fill method. The short section of the dam south of the outlet structure was placed by the rolled fill method, because of the small quantity required on that side of the river. A small portion of the yardage is represented by a blanket of impervious material, which was placed as rolled fill upstream from the cutoff trench, in places where the natural overburden was too thin to meet the requirements, and across the channel of the old river bed. Some puddled fill was placed in two cross dams which were required to confine a section of hydraulic fill. The excavation from the cutoff trench was placed in the outer toes of the main section of the embankment as was done at the other dams.

The outlet structure is very similar to the one at Taylorsville, although it was not quite as large. It contains 37,500 cu. yds. of concrete. The crest of the spillway is 100 feet long and the gap is spanned by a two-span reinforced concrete girder bridge. The outlets are three in number, each 16½ feet high and 15 feet wide. A highway 30 feet wide connecting Springfield Pike on one side and Valley Pike on the other crosses the valley on the crest of the dam.

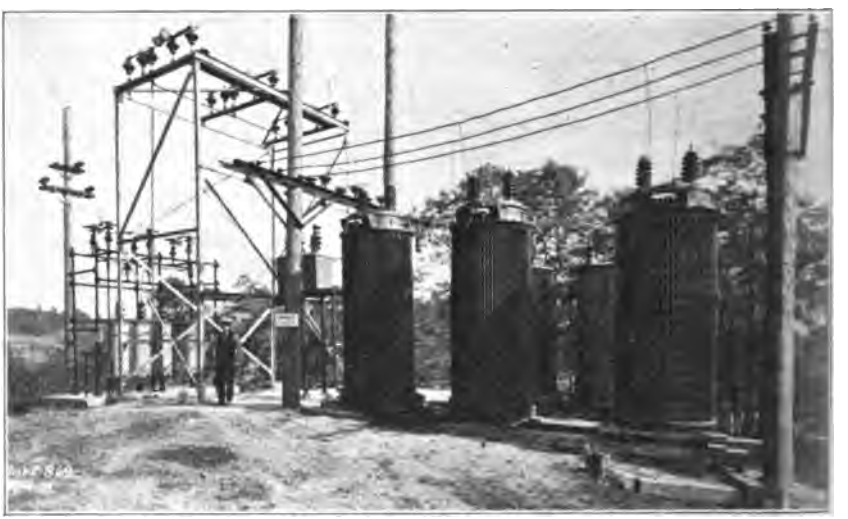


FIG. 30—TRANSFORMER STATION
This shows one of the installations of transformers and lightning arresters for power distribution at one of the dams.

The Big Four and Erie Railroads crossed the site of this dam, one on each side of the valley, not far above the general elevation of the valley floor. These railroads are operated under a cooperative agreement as a double track road, and in relocating them it was feasible to put them together on parallel tracks on the south side of the river. The Ohio Electric Railway also crossed the dam site on the south side of the river. The relocations of the Big Four and Ohio Electric were among the first things that had to be done at this dam site, as the old lines occupied the site of the outlet structure for the dam. The final relocation of the railroads necessitated a deep and extensive cut through the point of a hill, and extensive fills running both ways

from the cut. A couple of years would be required to complete the work. In order not to delay the construction work at the dam for that length of time it was decided to make a temporary relocation of the Big Four and the Ohio Electric, to get them out of the way of the outlet structure, so that the work on that structure could go on while the permanent relocation of the railroads was being finished. This temporary relocation could be accomplished in a few months time, and thus considerable time could be saved in the completion of the dam itself.

The railroad relocations were handled by contract under the supervision of the railroad company with the Miami Conservancy District suitably represented. The construction of the dam itself was handled by District forces, just as was done on the rest of the project, but the railroad contract provided that a certain amount of the outlet structure excavation at Huffman could be placed in the embankment for the relocated railroad.

Railroad Facilities. The two railroads and the electric line crossing this dam site furnished adequate transportation facilities to and from the work. Suitable sidings were installed connecting first with the existing lines and later with the new lines, when traffic was diverted to them. There were also two highways, paved for a part of the way, and in good passable condition at any season of the year, leading to the dam site from Dayton, the headquarters of the project.

Construction Program. As has been noted, the type of outlet structure at Huffman was the same as at Lockington and Taylorsville, therefore the stream control problem there was not difficult to handle. Mad River has the reputation of being a flashy stream, and much larger quantities of water must be handled there during a flood than at Lockington, but nevertheless, with this type of outlet structure, the stream control problem was not difficult to work out.

The necessary railroad relocations at Huffman, however, promised to be an important factor as affecting construction progress. Very little could be done on the outlet structure until the temporary relocation of the Big Four and Ohio Electric had been made. Only a certain amount of embankment could be placed to advantage until the Erie railroad, on the other side of the river, had been moved to its permanent relocation. So at Huffman the construction program had to be worked out with both stream control and railroad relocations in mind.

While the outlet structure was very similar to the one at Taylorsville, and the excavation problem somewhat the same, there was this difference, that at Huffman very little of the outlet excavation was suitable for use in the dam, therefore the hydraulic fill did not need to be carried on simultaneously with outlet excavation, as at Taylorsville.

The economical location for the outlet structure involved the occupation of a portion of the old river channel, so one of the first steps in the construction program at Huffman was to excavate a temporary cut-off channel through which the river might be diverted during the construction of the outlet works. The next step was to build the outlet works as soon as railroad relocation work would permit, and then to divert the river through them, as was done at the other dams, and to put in enough embankment during one low water season to close the channel and turn a maximum flood should one occur.

The construction program provided for the building of camps, installation of equipment, power lines, etc., during the spring of 1918 and was expected to continue as follows:—

For season of 1918.

(a)	Construction of temporary diversion channel.	
	Earth excavation	24,000 cu. yds.
(b)	Excavate for portion of outlet structure, and start placing concrete.	
	Earth excavation	43,000 " "
	Rock excavation	110,000 " "
	Concrete	20,000 " "
(c)	Build trestle across river to carry railroad and wagon roadway.	
(d)	Soil stripping	7,000 " "
(e)	Cut-off trench	5,000 " "
(f)	Place embankment along upstream toe of dam, high enough to carry railroad tracks and support hog-box for hydraulic fill.	
	Semi-hydraulic fill	45,000 " "
	Miscellaneous fill	16,000 " "

For season of 1919.

(a)	Complete excavation for outlet works, and finish concrete work, except spillway section and conduits.	

Earth excavation..... 5,000 cu. yds.
Rock excavation..... 30,000 " "
Concrete ..... 15,000 " "

(b) Soil stripping..... 21,000 " "

(c) Cut-off trench..... 12,000 " "

(d) Build section of dam between diversion channel and Erie railroad.
Hydraulic or semi-hydraulic fill.....510,000 " "
Semi-hydraulic or rolled fill.....150,000 " "
Miscellaneous fill..... 3,000 " "

For season of 1920.

(a) Make river closure and close gap at Erie railroad, bringing embankment to safe height for floods.
Hydraulic fill.....400,000 cu. yds.

(b) Complete excavation of outlet channel below outlet works.
Excavation ..... 20,000 " "

For season of 1921.

(a) Complete embankment.
Hydraulic fill.....270,000 cu. yds.
Miscellaneous fill..... 14,000 " "

(b) Complete outlet structure by building spillway section, conduits and spillway bridge.
Concrete ..... 2,500 " "

(c) Build roadway across crest of dam with guard rail, etc.

For season of 1922.

Finish, dress and seed slopes of dam.
Complete roadway and guard rails.
Clean up and finish all work by June.

Because of the complications due to railroad relocations, this original program was changed considerably before the dam was finished, but as far as the selection of equipment was concerned, the difference between the actual program and this preliminary one was not important.

Excavation Plant. The excavation for the Huffman outlet structure presented problems very similar to those at Taylorsville. The excavation was in Cincinnatian rock and amounted to 182,000 cu. yds. The bottom of the pit was all below river level,

and at the deepest part of the stilling pool it extended to a depth of 38 feet below low water elevation. This work must be protected against river flooding as at Taylorsville, and the same problem of track grades for cofferdam or dike protection presented themselves, as at Taylorsville, in connection with steam shovel work. A dragline excavator loading into 12-yd. dump cars was selected for the excavation plant at Huffman for practically the same reasons as decided the question at Taylorsville.

At Huffman, as at Germantown and Englewood, the most suitable material for hydraulic fill embankment was found in the river valley just upstream from the damsite. This was composed of river sand and gravel with an overburden of silt and loam very similar to material at Germantown and Englewood, except that the material was more porous having a less percentage of fines than was the case at the other places.

For this borrow pit excavation, a dragline machine and cars were selected for the same reasons as have been given for the selection of similar equipment at Germantown and Englewood.

After the work started it was found that there was not enough fine material in the Huffman borrow pits to give the required amount of core in the dam, and to meet this deficiency, a borrow pit was opened up in the glacial till on the hillside at the north end of the dam, this material being broken down by hydraulic giants and sluiced direct into the core.

Concrete Plant. The outlet excavation was started at the upstream end and was finished to full width, working downstream, just the opposite from the way it was done at Taylorsville, principally because the best location for the washing and screening plant was near the upstream end of the outlet structure. This concrete plant at Huffman differed in no material way from those erected at the other dams. The sand and gravel was obtained from the main borrow pit in the river valley where the embankment material was also obtained. This was loaded into 12-yd. cars by the small dragline machine listed under excavation equipment, and hauled to the plant by 40-ton standard gage dinkeys. The mixed concrete was poured into bottom dump buckets on narrow gage flat cars and hauled to the job by 3-ton gasoline locomotives. At the job these buckets were handled by two 105-foot boom stiff-leg derricks, mounted on a common base. These twin derricks were on skids and rollers and were moved along downstream as the work progressed. They were

used also for handling the wall forms which were made up into panel sections of convenient size as at Taylorsville. A substantial crib and trestle foundation was required to carry these derricks across the deep part of the stilling pool.

Hydraulic Plant. As has been said, the most suitable material for hydraulic fill at Huffman lay in the river valley just upstream from the damsite. As at Germantown and Englewood this ma-

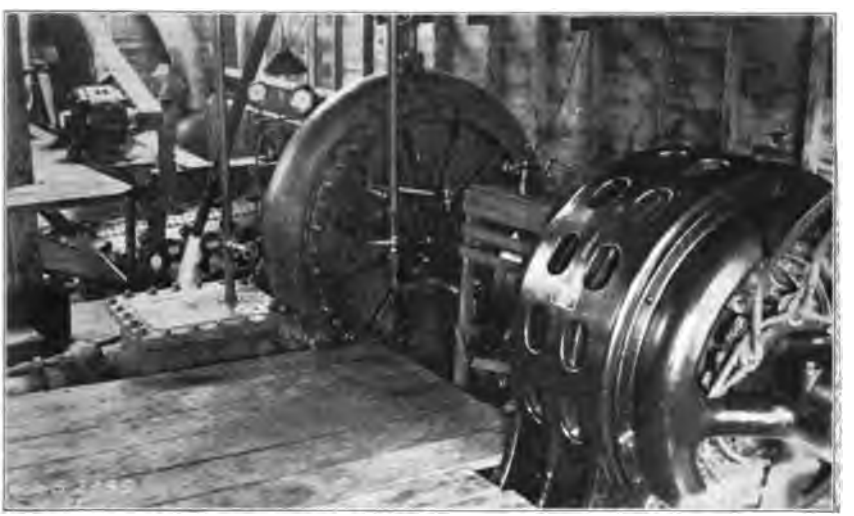


FIG. 31—DREDGE PUMP AND MOTOR

This is a 15-in. dredge pump connected to a 350-H.P. motor as installed at Huffman Dam. The revolving grizzly can be seen just beyond the pump. The hog-box is just outside the building at the right.

terial was loaded into cars by a dragline excavator and hauled to a hog-box, where it was picked up and pumped into place by means of dredge pumps. The best location for the hog-box at Huffman was in the upstream toe of the dam about midway across the valley. This location was not changed during the progress of the work. The high pressure pumping plant for the hog-box giants took its supply from the river, and the low pressure supply to the dredge pumps was taken in direct from the river by gravity. Fifteen-inch dredge pumps with 15-in. discharge lines were used here, as at most of the other dams, and

a booster pump was necessary in order to meet the requirements of friction and static head, to get all the material in place.

The discovery, after the work was well under way, that the borrow pit material was not furnishing a large enough percentage of fines to maintain the core dimensions as originally laid out, led to the opening up of a borrow pit on the hillside at the north end of the dam where an additional supply of fine material could be secured. This material was torn down out of the bank, and broken up, by a hydraulic giant under high pressure, and a great deal of it was sluiced direct into the core of the dam

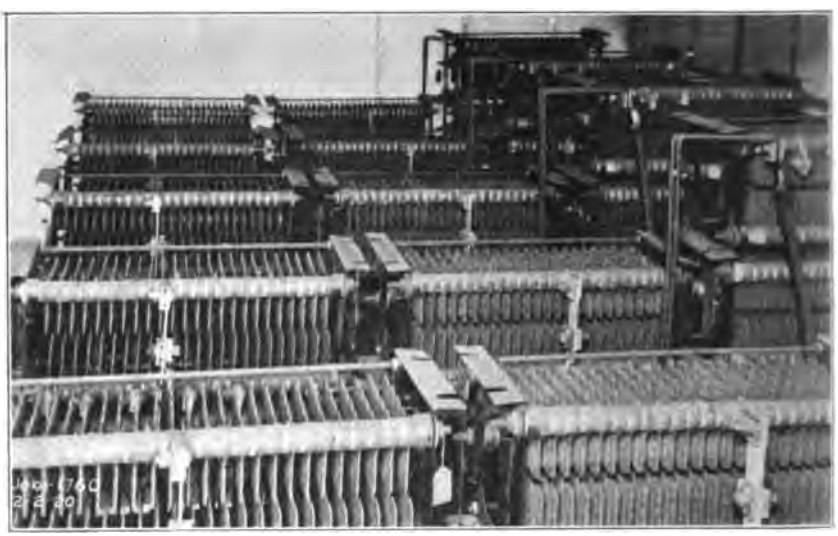


FIG. 32—RESISTANCE GRIDS FOR DREDGE PUMP MOTOR

through a long line of 15-in. pipe. This is the only case on the Conservancy work where it was feasible to sluice material direct from borrow pit to dam. As the embankment was raised and the higher portion of this hillside borrow pit worked out, it was necessary to resort to pumping in order to get into the dam all of this fine material that was required. This pumping was done by a 15-in. dredge pump, which was set into the sluicing line in such manner that it really acted as a booster pump and took advantage of all the head that could be obtained from the borrow pit. Additional high pressure pumps were required to serve the giants for this hillside borrow pit.

In the main borrow pit at Huffman, rocks and boulders were more troublesome than at any other dam. On the other hand, rock was required there for paving the upstream slope and was not easily available from any other source, so good use was made of all the oversize obtained at the hog-box. This oversize was separated out by means of the revolving grizzly as at Germantown and Englewood, and was put out into a storage pile by means of a stiff-leg derrick set up close to the hog-box. This derrick was placed in such position that it would also handle the dredge pumps and their suction lines when necessary, and was also available for cleaning out the sumps, if for any reason they became clogged.

Power Distribution Plant. The preliminary estimate of power requirements at Huffman was a total of 6,000,000 K.W.-hrs., with a maximum load of 900 K.W. The electric power was brought to the dam as "high tension energy" by means of a short transmission line, branching off from one of the Dayton Power & Light Company's main distribution lines. The usual substation equipment, and the additional transformers for distribution over the job are shown in the list and require no further explanation.

As to shop equipment and miscellaneous equipment the lists are easily understood, taken in connection with what has been said concerning similar equipment at the other dams.

Table XXI.—Equipment List—Huffman Dam

Note: Items purchased second hand are marked S.H. All others were new.

Excavating Equipment

		Price each
1	Bucyrus Class 24 electric dragline excavator, skid mounted, 85-ft. boom, 4½-yd. bucket, or 100-ft. boom, 3½-yd. bucket; main hoist engines driven by 200-H.P. type M Westinghouse motor; swing engine operated by 75-H.P. type M Westinghouse motor. Total working weight of machine 120 tons	$ 38,650.00
1	Lidgerwood Class B steam dragline excavator, truck mounted, 75-ft. boom, 1½-yd. bucket; hoist engine double cylinder 6½" x 8", Gem City boiler, 49-H.P. Weight of machine 50 tons. S.H.	13,500.00
1	American steam railroad ditcher; $\frac{9}{16}$-yd. bucket, standard upright boiler; equipped to operate clam-shell or dragline bucket	12,141.00
33	Western air dump cars; 12-yd. capacity, standard gage, side dump, 19-ft. body, 50,000-lb. capacity, complete standard M. C. B. equipment.	

		Price each
	12 @	1,600.00
	17 S.H. @	1,000.00
	4 @	1,930.00
3	American 40-ton locomotives; saddle tank, standard gage, type 040, cylinders 14" x 22", with M. C. B. complete and Westinghouse air equipment.	
	2 @	13,000.00
	1 S.H. @	9,500.00
1	Baldwin locomotive; saddle tank, standard gage, type 040, cylinders 15" x 24", steam brakes, standard M. C. B. equipment. S.H.	6,750.00
1	Western spreader car; standard gage, main blades 21-ft. x 4'-4", operated by hand winch	3,245.00
1	Standard rail laying car; standard gage	140.00
20700	ft. 60-lb. and 70-lb. rail; 700 ft. 30-lb. rail	9,750.00
1	Sullivan air compressor, class W.G. 3, size 10" x 10", single stage, 213 cu. ft. displacement at 100 lbs. and 235 R.P.M., belted to 40-H.P. Allis-Chalmers motor with speed of 1150 R.P.M. Vertical air receiver, 30" x 72"	1,438.00
1	Ingersoll-Rand wagon drill, "A" frame type, standard 20-ft. turntable, G-64 Dake hoisting and traction engine, 35-H.P. boiler, machine mounted on four flanged wheels. S.H.	1,500.00
1	Cyclone class B, well drill; non-traction, 24-ft. derrick, 7-H.P. steam engine and boiler	1,285.00
2	Ingersoll-Rand tripod rock drills, type E-44; air or steam operated with E-42 gland	275.00
3	Sullivan jap drills, class DP; solid piston, rotator type	155.00
1	Erie City 50-H.P. locomotive type boiler, mounted on trucks. S.H.	800.00
1	Caldwell No. 104 standard water tank, 16-ft. diameter, 13'-5" deep; made of Louisiana cypress	220.00

Concreting Equipment

1	Dull gravel washing and screening plant; 3 revolving inclined screens, 54" x 27" x 72" with ⅜", 1½" and 3" holes; 60" conical sand separator; 5-ply ½" x 20" x 350' conveyor belt; all operated by 20-H.P. Wagner motor; 3" American Well Works centrifugal pumps, direct connected to 25-H.P. Wagner motor. No bins or other structures included	2,319.00
2	Smith concrete mixers, No. 28-S; 1-cu. yd, capacity; on wood skids, power tilt, batch feeding hopper; driven by a 20-H.P. Allis-Chalmers motor	1,880.00
1	Smith concrete mixer, No. 7-S; ¼-cu. yd. capacity; power side loader; on trucks, with 5-H.P. Smith horizontal gas engine	776.00

	Price each
5 Lockwood concrete buckets; 1½-yd. capacity, double bail, bottom dump. S.H.	100.00
3 Clark concrete buckets; 1½-yd. capacity, double bail, bottom dump. S.H.	100.00
6 Sanford-Day flat cars; 36″ gage, Whitney roller bearing wheels, wood body, 3″ oak top, 55″ x 55″	202.00
2 Plymouth gasoline locomotives, model AL; 3-ton, 36″ gage	1,725.00
2 Bedford all-steel 8-ton derricks; 105-ft. boom, 40-ft. mast, 45-ft. stiff leg, 14-ft. bull wheel; operated by a Lidgerwood electric hoist, double drum friction type, with attached swinger, 52-H.P. General Electric motor	5,568.00
Hydraulic Equipment	
3 American manganese 15-in. dredge pumps; 115-ft. head, 4200 G.P.M.; direct connected to 350-H.P. Allis-Chalmers motor, 2200 volt, variable speed 350 to 514 R.P.M.	6,180.00
2 Hydraulic giants, size No. 2; ball bearing head, 2½″ and 3″ nozzles; made by Joshua Hendy Iron Works	206.00
1 Hydraulic giant, size 4; made in M. C. D. shops; 2½″ and 5″ nozzles	496.00
2 Allis-Chalmers 10″ centrifugal pumps, type "S"; double suction, single stage, 3500 G.P.M., at 180-ft. head, 1760 R.P.M.; direct connected to 200-H.P. Allis-Chalmers motor	2,200.00
1 Allis-Chalmers 8″ centrifugal pump, type "S"; double suction, single stage, 1950 G.P.M., 140-ft. head, 1750 R.P.M.; direct connected to 100-H.P. Allis-Chalmers motor	1,400.00
3500 ft. American Rolling Mills 15″ electrically welded, slip joint, dredge pipe. Special analysis. Including fittings	7,500.00
1500 ft. American 15″ asphalted spiral riveted pipe, with fittings	4,748.00
390 ft. 20″ slip joint dredge pipe. S.H.	75.00
450 ft. 3″ and 4″ wrought iron pipe	181.00
1 Revolving grizzly; 4-ft. diameter, 12-ft. long, with 7″ perforations; driven by a 7½-H.P. Allis-Chalmers motor	1,100.00
1 American 5-ton wood stiff-leg derrick; mast 38-ft. boom 56-ft, 12-ft. bull wheel; operated by American double drum hoist engine, 25-H.P. General Electric motor	1,320.00
Shop Equipment	
1 Famous 36″ band saw, Sidney Tool Co., No. 303; complete with tilting table	273.00
1 Oshkosh No. 1 rip and cross-cut table saw	319.00

	Price each
1 Superior 20" drill press; square base, wheel and lever feed, stationary head	141.00
1 Blount No. 6 emery grinder; floor type; suitable for two emery wheels	47.00
1 Royal No. 40 steel forge; 10" fan, 22" hearth, 30" high	27.00
1 Buffalo No. 3 steel pressure blower	18.00
1 Hay-Budden anvil; 195-lb.	37.00
1 Cyclone 20-ton chain hoist. S.H.	425.00
1 Yale & Towne 2-ton "Triplex" chain hoist	91.00
1 Allis-Chalmers 10-H.P. motor; 440 volts, 1150 R.P.M., type "AN"	272.50
1 American model A gasoline storage outfit, with measuring pump; 282-gal. tank	148.00

Power Distribution Equipment

3 General Electric transformers; 500 KVA, 33000/2300 volts	2,151.00
1 Packard transformer; 400 KVA, 33000/2300 volts, type "A"	1,517.00
1 General Electric galvanized steel tower for transformer station; type "T," 1617417, 15' x 9' base, 20' high, galvanized iron shapes	210.00
1 General Electric lightning arrester, catalogue 196481, 27000/37900 volts, sphere gap DL1500806; cone filler; set of copper tubing; 16-gal. K.D. Ammonia solution; type I, form C-4-D.O.	1,225.00
1 General Electric disconnecting sub-station switch; 35,000 volts, 300 amps., type L, form G19	285.00
3 Wagner transformers; 100 KVA 2300/460/230 volts, type HE, model 85-N	519.00
3 Wagner transformers; 75 KVA, 2300/460/230 volts, type HE, model 85	431.00
2 Wagner transformers; 10 KVA 2300/230/115 volts, type HE, model 53 N	107.00
1 General Electric transformer; 10 KVA, 2200/220/110 volts, type "H"	124.00

Note: All motors 3-phase, 60-cycle.
All transformers single phase, 60-cycle.

Miscellaneous Equipment

1 Orton-Steinbrenner 20-ton locomotive crane; steam operated, 45-ft. boom, 1-yd. clamshell bucket, 8 wheels; maximum reach 47-ft.; lifting capacity, 57,000 lbs. at 10-ft. radius, 5,200 lbs. at 60-ft. radius	21,000.00
1 Lambert hoisting engine; double drum, double cylinder 10" x 12"; with 35-H.P. vertical boiler	1,200.00
1 Vulcan steam pile driver, No. 2; 10½" cylinder, 36" stroke, 3,000-lb. striking weight, 50-ft. leads of 8" x 10" pine. S.H.	475.00

	Price each
1 Worthington 8" centrifugal pump, class "C"; belt driven, by 50-H.P. Allis-Chalmers motor	663.00
1 Worthington 6" centrifugal pump, class "C"; belt driven, by 30-H.P. Allis-Chalmers motor	498.00
1 Worthington 4" centrifugal pump, class "C"; belt driven by 20-H.P. Allis-Chalmers motor	388.00
1 Cameron piston pump; 7" x 6" x 7"; with 8-H.P. Gem City Boiler. S.H.	598.00
2 Weinman centrifugal pumps; 3" suction, 2" discharge, class "E," 80' head at 1750 R.P.M., 35-ft. head at 1150 R.P.M.; direct connected to 3-H.P. Allis-Chalmers motor	218.00
1 Weinman 3½" x 2¼" x 5" automatic steam pump and receiver	150.00
1 Gem City 20-H.P. horizontal boiler	609.00

The total of these equipment lists for the Huffman Dam is $274,200, which includes equipment items only, that is, those articles which have a value and use when considered apart from the plant as a whole, and which may be transferred from plant to plant, or used independently.

The *plant* cost at Huffman, including the above equipment, together with foundations, material for bins, temporary railroads and other construction which goes to make up a working plant, was.....$418,000.00

Add to this the cost of installation, (erecting bins, setting up equipment, assembling machines, etc.)..... 60,600.00

And the cost of dismantling..... 13,700.00

The total plant investment at Huffman was.....$492,300.00

All of which, however, was not necessarily on the job at one time, nor for the full construction period.

The total cost of the Huffman job was $1,880,000, of which the total plant investment was 26.2%. Of this total plant investment the cost of equipment items only, was 56%.

Dayton Channel Improvement

Description. The three main branches of the Miami River come together within the limits of the City of Dayton. The downstream section of the city was built on the flood plain of the river. On numerous occasions overflows on the valley floor have caused serious damage, but it remained for the flood of 1913 to show that the situation required serious attention and extensive work, to bring about adequate flood control.

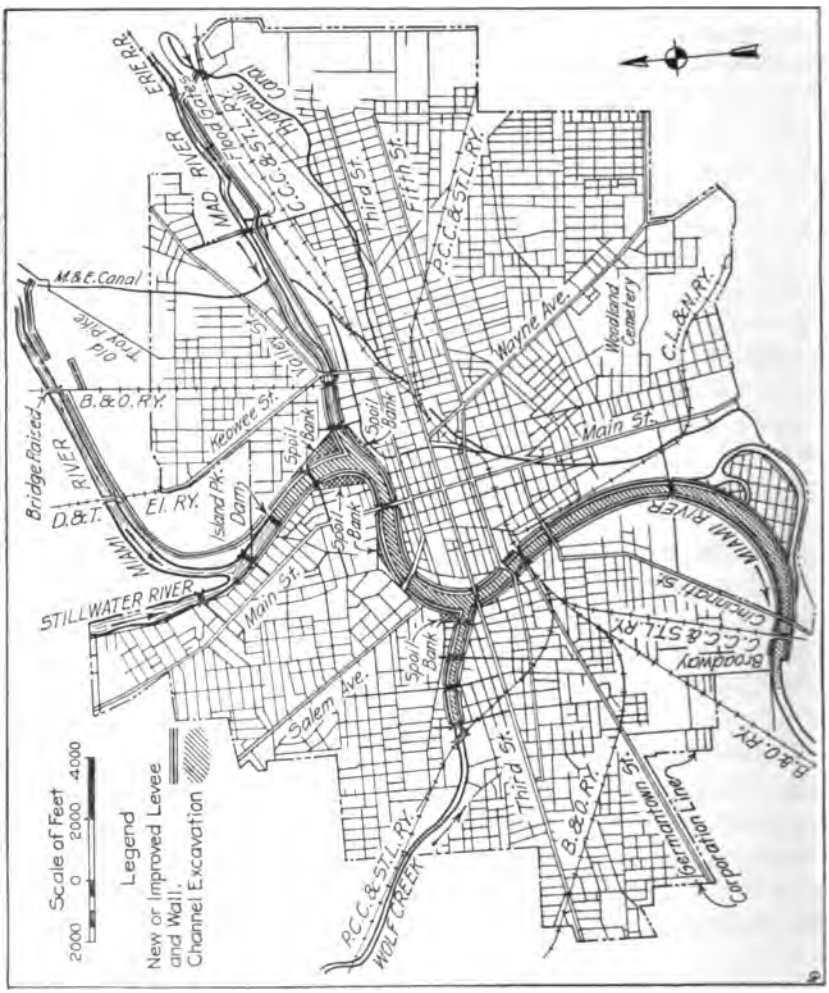


FIG. 33 – GENERAL MAP OF DAYTON CHANNEL IMPROVEMENT
Showing the river system, levee work, channel excavation and spoil banks.

The river takes a winding course through Dayton, as may be seen by a glance at Fig. 33. Its capacity was decreased materially by many islands and bars which had built themselves up in its bed, and by a rank growth of trees and brush within the limits of the river channel. Due to these obstructions, eddies and cross currents were caused during times of flood flow, which added further to the inefficiency and instability of the old channel. Within the limits of the city the channels were spanned by 22 bridges, the piers of which caused further obstructions to flood flow. Levees had been built along the channels at various points, and while these as a rule were substantially built, they were inadequate as to size, and were entirely submerged during the flood of 1913.

As has been mentioned in Chapter 1, channel improvement was the first remedy that suggested itself after the 1913 disaster. The conditions just described indicate that considerable relief could be obtained by cleaning out, straightening and deepening the river channel through Dayton (and similar conditions existed in other cities and towns). Widening of the river channel was also possible in some places, but ordinarily this was limited by industrial and other building developments, which made right-of-way for widening very expensive.

The original capacity of the river channel through Dayton was about 70,000 second-feet. It was found to be practicable to increase this capacity too about 120,000 second-feet by cleaning out, deepening, and straightening the alignment of the river channel, and by raising and enlarging the levees. This work involved the following quantities:

Channel excavation	2,150,000	cu. yds.
Embankment in levees	915,000	" "
Concrete in walls, revetment, etc	36,000	" "
Steel sheet piling	225	tons
Timber piles (along tooes of revetment	65,000	lineal ft.
Pipe lines lowered or relaid	6,500	" "
Levee dressing and seeding	280,000	sq. yds.

The work extended over a total river length of 6 miles, and the channel excavation was spread over an area of 650 acres, with an average depth of cut of only 3.6 feet. In the central portion of the city were long reaches where little or none of the excavation could be wasted in the banks adjacent, on account of the presence of buildings, trees, streets, or other obstructions. Much

of this material had to be hauled a distance of a mile or more to spoil banks. A large part of the levee embankment had to be placed under and around trees, or adjacent to buildings, walls, bridges or city streets. There were 13 separate pipe lines crossing the river, which had to be lowered below the limits of the improved channel. Sewer outlets and other obstructions along the river banks had to be removed, rebuilt, cut off or extended, to tie in with the slopes of the new channel. Exclusive of revetment work, concrete was to be placed in 101 separate structures in widely different locations.

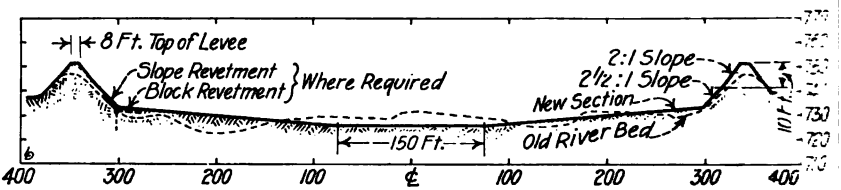


FIG 34—TYPICAL CROSS SECTION OF IMPROVED CHANNEL
The low water channel is 150 feet wide with gently sloping beaches on either side extending out to the toes of the levees.

Construction Program. There was no controlling factor on the river channel work, like the stream control at the dams, to dominate the program of construction. There was no reason why any of the channel improvement jobs could not be completed by the time the dams were finished, but even the partial protection afforded by enlarged channels was desirable as soon as it could be had, so the general plan was to start the two largest channel improvement jobs (Dayton and Hamilton) as early as possible and push them to completion with as much speed as was consistent with economy. This called for the assembling and installation of equipment for the Dayton job during the spring of 1918 and the completion of all work by September, 1921.

Excavation Plant. Various types of excavating equipment were considered for this work. Those which were given the closest study were steam shovel and cars; dipper dredge and cars or scows; slack line cableways and cars or scows; suction dredge, and dragline excavator.

Steam shovel and cars were not suitable for several reasons, the two principal ones being that much of the excavation must come from below water level, and that there was no room between levees in many places where tracks could be located so as to be protected from damage and undermining by frequent small freshets. Dipper or ladder dredges would be handicapped by insufficient depth of water for flotation, and would be unable to reach the material outside the center of the channel because of the flat beaches extending from the limits of the low water channel out to the toes of the levees, which were the requirements of the new channel section. See Fig. 34.) Slackline cableways were not suitable for general use because of lack of operating space along the banks, and for other reasons. The character of the material was not suitable for handling by suction dredge, and because of the shallow cuts a suction dredge would be continually at a disadvantage; moreover, the difficulty of reaching the material outside the center of the channel would apply to suction dredges the same as to dipper or ladder dredges. A plan was studied involving the use of hydraulic giants to cut down the banks and trim down the channel slopes, sluicing the material to within reach of floating dredge equipment, but this was never worked out satisfactorily.

Dragline machines, however, answered most requirements of this job fairly well. They could handle the wet excavation without difficulty, and their long reach would enable them to clean up the material along the beaches without an unreasonable amount of moving. They could load material into cars or scows, and could quickly move to a place of safety in case of a flood, in fact, they would stand a certain amount of flooding without inconvenience, and would be ready to start working again immediately after the flood had receded. A situation of that sort is illustrated by Fig. 35. The dragline machine could even be mounted on a scow and perform in a fairly effective manner such of the work as needed to be handled by floating equipment. In Chapter II, under the heading "Dragline Dredge" is a detailed account of the use of the dragline machine under such conditions. In places where the material could be put direct into levee or waste bank alongside the excavation, the dragline machine had an advantage over all other types of equipment, because of its long reach and effectiveness under such conditions. Even where two or three throws would be necessary, to get the material from cut

to bank, a dragline machine would be more economical than any other type of equipment. It was for these reasons, therefore, that dragline machines were selected as the principal items of excavating equipment for this job. Machines with large capacity and long reach were to be preferred for the main body of the excavation. The Class 175 Bucyrus answered these requirements as they could be depended upon to handle 70,000 to 80,000 cu. yds. per month under reasonably favorable conditions, and

FIG. 35—DRAGLINE MACHINE IN FLOOD

These large machines stand so high that they can withstand a considerable flood without damage to motors or boilers.

when equipped with 135-foot boom could reach as far as 250 to 275 feet between excavation and dump. The Lidgerwood machine shown on the list, was brought on to this work to help finish it, after the completion of another job.

A small machine was needed to handle parts of the work where the large ones would be too big and unwieldy, for work around bridges, moving pipe lines, and many small sections of work where shallow cutting and frequent moving gave a small machine

the advantage. Caterpillar traction was desirable for this lighter equipment, for obvious reasons.

Transportation to spoil banks at the upper end of the work could not well be handled by trains, as there was no feasible location for a track, but it was possible to work out a satisfactory plan by using scows and tug boat, loading the scows by a dragline machine mounted on a spud scow or working from the shore. The three deck scows shown in the equipment list were for transporting material; the large spud scow was for the large dragline machine, and the small spud scow was used for miscellaneous work, part of the time to serve as a float for the small caterpillar dragline. All of this floating equipment was built on the ground.

Cars could be used to advantage on certain sections of the work. The larger locomotives were obtained for hauling the 12-yd. cars, and the small steam dinkey and the gasoline locomotive were for use with the 4-yd. cars, and the small dragline.

Concreting Equipment. The amount of concrete work done on the Dayton job justified the erection of a gravel plant, particularly because the supply of gravel at that time from commercial plants was hardly sufficient to meet the local demands. In fact, an arrangement was made, as soon as the Conservancy plant was erected, to dispose of all its surplus output through one of the local gravel companies. As the concrete work at Dayton need not, and in some places could not, start until the excavation was partly completed, it was arranged to transfer the gravel plant from Germantown Dam to Dayton as soon as the work at the dam would permit.

Because the work at Dayton was so scattered, however, this plant was set up simply as a washing and screening plant, it being arranged to haul the prepared sand and gravel to the various jobs and run through their mixing plants directly into place. The river excavation would furnish an ample supply of suitable material for this plant and it was planned to obtain this material mostly from the section where floating equipment was used for excavation, then the material could be brought to the plant on scows. Two guy derricks were required, one to unload the material from the scows, and pass it to the plant, the other to place the surplus output in storage piles temporarily until it might be disposed of.

A number of mixers were required for the various concrete jobs. One or two of the retaining wall jobs were large enough to

require the use of a 1-yd. mixer. The ½-yd. mixer on trucks was a convenient size for some of the other work, while many of the small scattered jobs required the use of the smaller ¼-yd. mixers. An important item of the concrete work at Dayton was the block paving, and about 400,000 of these special blocks would be required for that purpose. It was expected, however, to sublet the job of making those blocks as it was work that could well be handled by a local contractor with suitable facilities. No special equipment, therefore, was ordered for this block work, although when the contract was finally made, it was arranged that such of the District's equipment that was suitable could be used on the work at an agreed charge for such use.

Power Distribution Equipment. The electric power for this job was taken from the power company's high tension lines at one or more points convenient to the work, and from the main power station of the company which was located on the river near the lower limits of this job. The power obtained at the plant was taken at 6,700 volts; at the other points it was 33,000-volt current. Suitable transformers for stepping down this current, and other equipment required at the transformer station, and for distribution to the various units of construction plant, are included in the equipment list.

Shop Work. With the exception of a few places where a forge and anvil might be set up temporarily, it was expected that all the shop work for the Dayton job would be taken care of at the main shop, therefore no special shop equipment for this job was provided.

Miscellaneous Equipment. The list of miscellaneous equipment includes a number of unwatering pumps, of which many were required, because of the large number of small structures, most of them requiring some wet excavation for foundation. Three stiff-leg derricks were provided also, to handle some of the structures, like retaining walls, where a derrick was particularly fitted to handle the excavation, the forms and the concrete. The necessary equipment of pile drivers was also provided as will be seen from the list.

Table XXII.—Equipment List—Dayton Channel Improvement

Note: Items purchased second-hand are marked S.H. All others were new.

Excavating Equipment

		Price each
2	Bucyrus electric dragline excavators, class 175, mounted on propelling trucks, 125-ft. boom, 3½-yd. bucket; main hoist operated by 250-H.P. Westinghouse motor, speed 505 R.P.M.; swing engine operated by 125-H.P. Westinghouse motor, speed 690 R.P.M.; total working weight 173 tons. S.H.	$ 50,000.00
1	Lidgerwood Class K electric dragline excavator, skid mounted, 100-ft. boom, 2½-yd. bucket; main hoist engines driven by 112-H.P. General Electric slip ring motor; swing engine operated by 52-H.P. General Electric slip ring motor; total working weight 110 tons. S.H.	20,833.00
1	Bucyrus steam dragline excavator, class 9½, mounted on caterpillars, 45-ft. boom, 1½-yd. bucket, double cylinder main engines, 8" x 8"; double cylinder swing engine, 5" x 5"; boiler, 50" x 10'; total weight 52 tons. S.H.	15,500.00
10	Western air, side dump cars; 12-yd. standard gage, 19-ft. wood bodies, 50,000 lbs. capacity, complete M.C.B. and Westinghouse equipment	1,930.00
2	American 40-ton locomotives; saddle tank, standard gage; cylinders 14" x 22"; type 040; complete with M.C.B. and Westinghouse equipment	13,000.00
1	Baldwin 35-ton locomotive; standard gage, saddle tank, type 040, 15" x 24" cylinders, steam brakes. S.H.	6,750.00
10	Western dump cars; 4-yd., 36" gage, wood bodies, M.C.B. equipment	301.00
1	Vulcan 15-ton locomotive; 9" x 14" cylinders, saddle tank, 36" gage. S.H.	2,300.00
2	Plymouth 6-ton gasoline locomotives; model BL, 36" gage, with electric lights and starter	3,040.00
8000	ft. 60-lb. and 70-lb. rail; 5000 ft. 30-lb. rail	3,625.00
1	Spud scow (M. C. D. shops), 40-ft. x 80-ft. x 6-ft.	
3	Deck scows (M. C. D. shops), 40-ft. x 120-ft. x 5-ft.	31,500.00
1	Spud scow (M. C. D. shops), 8-ft. x 18-ft. x 2.2-ft.	
1	Steam tug, sternwheeler; built by M. C. D.; driven by pair of 8" x 42" link motion piston valve engines; capstan operated by 5" x 5" vertical duplex engine; Gem City locomotive boiler, 48" x 15-ft.; capable of towing 115 yds. of wet gravel on 40-ft. x 120-ft. x 5-ft. deck scow	16,000.00

Concreting Equipment

1 Dull gravel washing and screening plant; three revolving inclined screens, 54" x 27" x 72" with ⅜", 1½" and 3" holes; 60" conical sand separator; 5-ply,

	Price each
½" x 20" x 350' conveyor belt; all operated by 2-H.P. Wagner motor. Water supplied by 3½" American Well Works centrifugal pump, direct connected to 25-H.P. Wagner motor. No bins or other structures included	2,319.00
1 Smith concrete mixer; 1-yd. capacity, size 28-S, mounted on wood skids, power tilt, batch feeding hopper, driven by 20-H.P. Allis-Chalmers motor	1,880.00
1 Smith concrete mixer, ½-yd. capacity, mounted on trucks, power side loader; with 10-H.P. Smith steam engine and 12-H.P. boiler	2,027.00
1 Lakewood concrete mixer, ½-yd. capacity, style 24, mounted on trucks, power side loader, Lakewood engine 6½" x 7½" reversing type, vertical boiler. S.H.	1,200.00
2 Smith concrete mixers, ¼-yd. capacity, size 7, mounted on trucks, power side loader; operated by 5-H.P. Smith horizontal gas engine	776.00
1 Western jaw crusher; 9" x 16", weight 5-ton, capacity 12 to 18 tons per hour. S.H.	631.00
1 Austin No. 4 left-hand gyratory rock crusher; manganese head, concaves to crush 1½" ring size; belted to a 50-H.P. Westinghouse motor	4,193.00
6 Sanford Day concrete cars, 1½-yd. capacity, 36" gage, radial gate side dump	206.00
1 Plymouth gasoline locomotive, model AL; 3-ton, 36" gage	1,725.00
3 Insley concrete buggies; 6 cu. ft. capacity. S.H.	19.00
1 Huber water tank wagon; 450 gal.	225.00
2 Bedford 8-ton all-steel derricks; 105-ft. boom, 40-ft. mast, 45-ft. stiff legs, 14-ft. bull wheel; one operated by a Lidgerwood electric hoist, double drum friction type, with 52-H.P. General Electric motor, with auto brake and attached swinging gear. The other operated by a Mundy 8" x 12" double drum hoisting engine and a Lidgerwood 6-H.P. reversible swing engine 5" x 12" double cylinder, 30-H.P. Mundy boiler. S.H.	4,760.00

Power Distribution Equipment

2 General Electric 400 KVA transformers, 33,000 to 2,300 volts, type H, form VDD	1,766.00
4 Wagner 200 KVA transformers, 6600 to 2200 volts, type HE	869.00
9 Wagner 100 KVA transformers, 2200 volts primary, 460/230 volts secondary, type HE	519.00
6 Westinghouse 75 KVA transformers, 2400 volts primary, 480/240 volts secondary, type S	450.00
4 Wagner 75 KVA transformers, 2200 primary voltage, 460/230 secondary voltage, type HE	430.00

Miscellaneous Equipment

		Price each
1	Wood stiff leg traveling derrick on skids with 24 8" rollers, 1-yd. Owen clamshell bucket, 62-ft. boom, 24-ft. mast, 12-ft. bull wheel; operated by a Buffalo hoist engine, double cylinders 9" x 10", three drums with winch heads, and Buffalo swing engine, double cylinder 4½" x 5", 35-H.P. vertical boiler 48" x 8-ft. S.H.	4,300.00
1	Wood stiff leg traveling derrick on skids and rollers; 1-yd. Owen clamshell bucket, 62-ft. boom, 27-ft. mast, 12-ft. bull wheel; operated by Rawson-Morrison three drum hoist engine, double cylinder 8¼" x 10"; swing engine double cylinder 4½" x 6", and 30-H.P. vertical boiler. S.H.	4,000.00
1	Wood stiff leg derrick with 1-yd. Owen clamshell bucket; 65-ft. boom, 30-ft. mast, 12-ft. bull wheel; operated by Lambert double drum hoist engine, double cylinder 10" x 12", National single drum swing engine, double cylinders 5" x 8", and a 35-H.P. vertical boiler. S.H.	3,300.00
2	Vulcan steam pile drivers, No. 2, 10½" cylinder, 36" stroke, 3000-lb. striking weight, 50-ft. leads of 8" x 10" pine.	
	1 @	600.00
	1 S.H. @	350.00
1	Vulcan steam pile driver, No. 2, 10½" cylinder, 36" stroke, 3000-lb. striking weight, 50-ft. leads of 8" x 10" pine. S.H.	475.00
1	Ingersoll-Rand steam sheet pile driver, type G-1; 3" jaw, 4" cylinder, 300 strokes per minute, 7 to 12 H.P., total weight 1375 lbs. S.H.	300.00
1	Mundy hoisting engine and boiler, double cylinders 8" x 12", double drum, with jaw clutch and winch heads. S.H.	1,000.00
1	American hoisting engine, double cylinder, 5½" x 8", 2 10" drums and winch heads, with 20-H.P. boiler. S.H.	320.00
1	Platt 14" centrifugal pump, double suction, single stage, split case, 30-ft. head, 865 R.P.M., 1000 G.P.M., direct connected to 50-H.P. Westinghouse motor. S.H.	1,274.00
1	Swaby centrifugal pump; 10" suction, 8" discharge; direct connected to vertical steam engine. S.H.	150.00
1	Worthington low head horizontal centrifugal pump, size 8, class C; 10" suction, 8" discharge; belted to a 50-H.P. 1150 R.P.M. Allis-Chalmers motor	663.00
1	American low head centrifugal pump; 8" suction, 6" discharge; direct connected to upright steam engine. S.H.	225.00
1	Worthington centrifugal pump; 8" suction, 6" discharge, class C; belt driven by 30-H.P. Allis Chalmers motor	498.00

	Price each
1 Worthington centrifugal pump; 6" suction, 4" discharge, class C; belt driven by 20-H.P. Allis-Chalmers motor..	388.00
1 Weinman centrifugal pump; 4" suction, 3" discharge, 125-ft. head, 220 G.P.M.; direct connected to 25-H.P. Allis-Chalmers motor	540.00
2 Weinman centrifugal pumps; 3" suction, 2" discharge, class E, 80-ft. head at 1750 R.P.M., 35-ft. head at 1150 R.P.M.; direct connected to 3-H.P. Allis-Chalmers motor	218.00
1 Weinman vertical sump pump, size 2; 3" suction, 3" discharge, 10-ft. head, 1130 R.P.M., 200 G.P.M.; equipped with automatic ball float; direct drive by 2-H.P. Allis-Chalmers motor	337.00
2 Weinman centrifugal pumps; 3" suction, 2" discharge, class E, 80-ft. head at 1750 R.P.M., 35-ft. head at 1150 R.P.M.; direct connected to 3-H.P. Allis-Chalmers motor	218.00
2 Domestic No. 4 diaphragm pumps; 4" discharge; operated by 2-H.P. Domestic gasoline engine; mounted on common base and four wheel truck	220.00

The total cost of the equipment items for the Dayton work was $313,600, including only those articles which have heretofore been described as equipment, as distinguished from plant.

The cost of *plant* for the Dayton work, including these equipment items, together with foundations, temporary railroads, temporary buildings and similar non-transferable parts of a working plant, was	$435,000.00
Add the cost of installation (setting up equipment and assembling machines)	80,400.00
And the cost of dismantling	15,100.00
Making the total plant investment	$530,500.00

All of which, however, was not necessarily on the job at one time, nor for the full construction period.

Total cost of the Dayton work was $2,270,000, of which the total plant investment was 23.4%. Of this total plant investment the value of equipment items only, was 59%.

Hamilton Channel Improvement

Description. At Hamilton was another large job for channel improvement which compared in size with the work at Dayton, and required one of the early plant installations. The river channel at Hamilton had been encroached upon severely by in-

dustrial plants to the extent that considerable widening had to be done there in spite of the great expense.

To a limited extent, levees had been built along the channel at certain points following previous floods, but this work had been entirely inadequate when the 1913 flood came along. A very large portion of the city was inundated.

The original capacity of the river channel through Hamilton (at the time of the 1913 flood) was 100,000 second-feet. The peak flow during the 1913 flood was about 350,000 second-feet.

FIG. 36—60-TON LOCOMOTIVE

To check up with the retarding basin control for the rest of the valley, it was necessary and practicable to increase this channel capacity to about 200,000 second-feet, principally by deepening and widening the channel and bringing it to regular lines, together with some levee construction where needed. Higher velocities would be required here than at other sites, therefore the paving requirements were larger. The work as planned involved the following quantities:

Channel excavation	2,243,000	cu. yds.
Embankment in levees	378,900	" "
Concrete in walls, revetment, sewers, etc.	21,970	" "
Steel sheet piling	247	tons

Timber piles	59,125	lineal ft.
Levee dressing and seeding	154,000	sq. yds.
Reinforcing steel	550,000	lbs.

The work extended over a total river length of about four miles. A comparatively small amount of channel excavation could be disposed of in levees alongside, but the greater portion of it had to be hauled to waste banks, two of which were established, one at the upper end of the work and one near the lower end. The average haul to these waste banks was approximately one mile. The average depth of cut was considerably more than it was at Dayton. There were no pipe lines across the river to complicate the work, but the sewer outlets and other structures along the river banks had to be moved, rebuilt or modified, to fit the new channel, as was the case at other channel improvement jobs.

Construction Program. The only requirement here as to construction progress, was to get the work started promptly, so that it could be completed as soon as possible, and at least by the time the dams were finished. The plan was to get the plant assembled and installed during the spring of 1918, and to so conduct the work that it would be completed in May 1921. After the work was started, it was arranged that the District would handle for the city the construction of the Black Street bridge, and later it was decided to build a dam at the upper end of the city to stop the drifting of gravel down through the channel, therefore the construction program as originally planned was modified, and the date of final completion was postponed somewhat.

Excavation Plant. As has been said, it was necessary that a large portion of the excavation at Hamilton be hauled some distance to waste banks. The layout was such that a construction track could be built within the limits of the improved river channel so as to be reasonably free from damage by small floods, so locomotives and cars for transporting equipment were decided upon without much question. As to excavating equipment the choice was between steam shovel and dragline machine. Either would be suitable for a large portion of the work and could handle it economically, but for the material that lay below water level the steam shovel would hardly be suitable. There were many places where the long reach of the dragline was a great advantage, and in locating the construction tracks, this advantage was very marked, because it permitted a location farther away from the low water channel, and therefore less liable to

interference by floods, than would be possible with a steam shovel. The dragline machine would also be less liable to damage to itself, by moderate floods in the river, because it could work from higher levels, and could stand deeper flooding than a steam shovel could. For these reasons the dragline machine was decided upon as being best fitted for this work.

A machine with a boom about 100 feet long and carrying a 4 to 5-yard bucket was considered to be the right size for the main excavation. To assist this machine, and for building track grades, and for work around bridges and other miscellaneous uses, two 60-foot boom machines were selected, one on skids and rollers, and one on caterpillar traction. Another small machine, a Marion Model 21, on caterpillars, with 30-foot boom, was secured, principally for the purpose of shifting track on the dumps, unloading cars and other miscellaneous uses.

Twelve-yard standard gage dump cars hauled by 50-ton saddle tank locomotives were chosen for the hauling equipment, and a spreader car was obtained for use on the dumps.

The small scows listed under excavating equipment were used for shifting pumps along the river and other miscellaneous uses, and an air compressor and drills were required for drilling bridge piers and other solid structures encountered during the excavation.

Concreting Equipment. The concrete work at Hamilton, as at Dayton, was scattered over a large area, and consisted of many different structures, mostly walls, sewer outlets, etc. It was intended to sublet the slope revetment here as well as at the other cities, so no special equipment was obtained for that.

A portable gravel screening plant of small capacity was used at first for supplying sand and gravel for concrete, but later a local commercial plant was put up, and as soon as that was put into operation, most of the necessary material was obtained from that plant. The mixers consisted of one 1-yard machine, two ½-yard mixers, and one of ¼-yard capacity. In some places the mixed concrete was handled by the traveling derricks listed under miscellaneous equipment, and in other cases it was loaded into rocker dump cars, hauled by 3-ton gasoline locomotives.

Power Distribution Equipment. The electric power for this work was furnished by a local company at 2300 volts, and was distributed to the various parts of the work through transformers furnished by the District.

Shop Equipment. A shop was required at Hamilton similar to those installed at the dams, and the equipment required for it was similar to that at the other places, as will be seen from an inspection of the list.

Miscellaneous Equipment. Under this heading is included the unwatering pumps required for the various items of work, movable stiff-leg derricks used for excavating and concreting walls and other structures, pile drivers, etc.

Black Street Bridge. During the 1913 flood the bridge over the river at Black Street was carried away. A suspension foot bridge was substituted, and had been in use since that time. Conditions were shaping up so that the City of Hamilton was about ready to take up the construction of a new bridge, but had not yet seen the way to finance it.

The Conservancy work required a considerable widening of the river at this point. The Conservancy District had the equipment and the organization to build a bridge economically, and it was found that the interest of the District, and of the City of Hamilton, would be best served by constructing a new bridge before the work of the District in that vicinity was undertaken. By such procedure the foot bridge could remain in service until the new bridge was open to traffic, and the construction work on the bridge would not interfere with the other work of the District. So an agreement was made between the city and the District whereby the District would build the bridge for a certain fixed sum, this cost to be added to the Conservancy assessment against the City of Hamilton, and collected as a part of the Conservancy tax.

The new structure is a seven-span concrete arch bridge, 708 ft. long, with 28-ft. roadway and two 6-ft. sidewalks. Each arch has a clear span of 86 ft. The piers are 58 ft. long and 7 ft. thick at the top. The maximum depth of the footings is 16 ft. below the proposed bed of the channel. These footings rest on piles which were driven to an average depth of 17 ft. below cutoff. The estimated quantities of the principal items of work were as follows:

Excavation	22,000	cu. yds.
Backfilling	7,500	cu. yds.
Timber piles	1,600	lin. ft.
Concrete	9,600	cu. yds.

The construction plant for this bridge was mostly on the ground, in use on other parts of the work. The excavation for foundations was made by a Class 14 Bucyrus steam dragline excavator. A description of this feature of the work, which was rather unusual, will be found in Chapter II, under the heading, "Building Bridge Piers." That describes also the handling and driving of piles, and the placing of concrete in the pier foundations, all of which was handled by this dragline machine. The pumping equipment consisted of a 4-in., 6-in., 8-in. and 12-in. pump, driven by 20-H.P., 30-H.P., 50-H.P. and 75-H.P. motors respectively. Each motor was belt-connected to its pump, and each unit so formed, was mounted on one base and housed in. These pumping units were also easily handled by the dragline machine when necessary to move them from place to place, or to lift them out of danger from high water. For the pile driving a Vulcan No. 3 steam hammer was used in timber leads 40 ft. long, which were hung from the end of the dragline boom. Steam for the hammer was supplied from the dragline boiler.

The only special equipment obtained for this bridge work was the Lidgerwood cableway, listed under "Concrete Plant." The many sudden floods to which the Miami River is subject, especially during the winter and spring, made it very undesirable to depend for concrete distribution on a trestle, or other equipment which might be in danger from high water or drift. Not only danger to the equipment itself, but the more serious danger of delay to the work, made it most desirable to select equipment which would be free from flood danger. For these reasons, and because one was available at a favorable price, the cableway was selected. It was used not only for placing concrete, but for transporting forms, and form lumber, reinforcing steel, waterproofing, and, in fact, practically all materials used in the construction of the bridge. The concrete mixing plant was located in the excavation for the east abutment. It consisted of a 1-yard Smith mixer, driven by a 20-H.P. motor, gear-connected. A batch hopper was built directly above the mixer, and a sand and gravel hopper above that. A 24-inch gage track connected the sand and gravel hopper with the storage pile, which had been built up out of the river excavation by the dragline machine. The traveling derrick, with clamshell bucket, handled this material to the screens, or to the cars.

Table XXIII.—Equipment List—Hamilton Channel Improvement

Note: Items purchased second-hand are marked S.H. All others were new.

Excavating Equipment

		Price each
1	Bucyrus class 24 electric dragline excavator; skid mounted, 100-ft. boom, 3½-yd. bucket; main hoist engine driven by 200-H.P. type "MW" Westinghouse motor; swing engine operated by 75-H.P. type "MW" Westinghouse motor; total working weight 120 tons. S.H.	$ 31,500.00
1	Bucyrus class 14 steam dragline excavator; mounted on caterpillars, 60-ft. boom, 2-yd. bucket; main engine double cylinder 8" x 10"; swing engine double cylinder 5½" x 6"; boiler 54" x 9'-7"; total working weight 75 tons. S.H.	18,000.00
1	Bucyrus class 14 steam dragline excavator; skid mounted, 60-ft. boom, with 15-ft. extension, 1½-yd. bucket; main hoist engines 8" x 10" double cylinder; swing engines 5" x 6" double cylinder; boiler 54" x 9'-6"; total working weight 75 tons. S.H.	15,500.00
1	Marion model 21 steam dragline excavator; mounted on caterpillars, 32-ft. boom, ¾-yd. bucket; main engine double cylinder 6" x 6"; swing engine double cylinder 4½" x 5"; 24 to 34-H.P. vertical boiler, 45⅝" x 96"; total working weight of machine 26 tons	9,650.00
38	Western air dump cars; 12-yd. capacity, standard gage, side dump, 19-ft. wood body, 50,000-lb. capacity trucks, complete standard M. C. B. equipment.	
	30 @	1,625.00
	8 @	1,500.00
1	Western spreader car; standard gage, 21-ft. blade, operated by hand winch, 60,000-lb. capacity trucks, standard M. C. B. construction. S.H.	1,500.00
1	Rail laying car, standard gage	140.00
3	American 50-ton locomotives; saddle tank, type 040, standard gage, cylinders 16" x 24", complete M. C. B. and Westinghouse air equipment	15,000.00
1	American 40-ton locomotive; saddle tank, type 040, standard gage, cylinders 14" x 22", complete M. C. B. and Westinghouse air equipment	11,200.00
22400	ft. 60-lb. rail (and accessories)	14,214.00
1	Scow, 4.5-ft. x 29.5-ft. x 51-ft. S.H.	175.00
1	Scow, 3-ft. x 9.5-ft. x 40-ft. S.H.	50.00
1	Caldwell No. 104 standard water tank; 16-ft. diameter, 13-ft. 5-in. deep ; made of Louisiana cypress	220.00
1	Huber water tank wagon, 450-gal.	225.00
1	Sullivan air compressor; class WG-3, size 10" x 10", single stage, 213-cu. ft. displacement at 100-lb. and	

		Price each
	235 R.P.M.; belted to 40-H.P. Allis-Chalmers motor with speed of 1150 R.P.M.; vertical air receiver, 30" x 72"	1,438.00
2	Sullivan handhammer drills, class DP-33; solid piston rotator	137.00
	Concreting Equipment	
1	Western portable gravel screening plant; 25-yd. bin, 4 compartments; 8-ft. x 32-in. revolving screen for ⅜", 1½" and 3" mesh, 28-ft elevation on rubber belt, operated by 7½-H.P. Allis-Chalmers 1200 R.P.M. motor	1,630.00
1	Smith concrete mixer; 1-yd. capacity, size 28S, mounted on skids, with batch loading hopper and power tilt; direct drive to 20-H.P. Allis-Chalmers motor	1,880.00
2	Smith concrete mixers; ½-yd. capacity, size 14S, mounted on 4-wheel truck, power tilt and power side loader; 10-H.P. Smith steam engine, 12-H.P. boiler	2,027.00
1	Smith concrete mixer; ¼-yd. capacity, size 7S, mounted on 4-wheel truck, with power side loader; driven by Smith 5-H.P. horizontal gasoline engine	776.00
6	Western all-steel dump cars; 1½-yd. capacity, V-shaped, side dump type, 24" gage	75.00
1	Plymouth gasoline locomotive, model AL; 3-ton, 36" gage	1,725.00
2900	ft. 30-lb. rail and fastenings	1,392.00
2	Lakewood concrete buckets; 1½-yd. capacity, double bail, bottom dump. S.H.	100.00
1	Lidgerwood cableway; operated by 9 x 10 D.D. D.C. hoist engine and 50-H.P. Gem City locomotive type boiler; main cable 1¾", 1366 ft. long; head and tail tower 80 ft.; made up of 12 x 12 yellow pine with oak head blocks	7,500.00
1	Wallace No. 5 bar bender, for reinforcing steel	148.00
	Power Distribution Equipment	
3	Packard 100 KVA transformers, 2300 volts primary, 440 volts secondary	533.00
6	Westinghouse 75 KVA transformers, 2200 volts primary, 440/220 volts secondary, type "S"	450.00
1	Wagner 75 KVA transformer, 2200 volts primary, 440/220 volts secondary, type "HE"	430.00
2	Wagner 10 KVA transformers, 2200 volts primary, 220/110 volts secondary	106.00
4	Westinghouse 5 KVA transformers, 2300 volts primary, 230/115 volts secondary	78.00

Note: All motors 3-phase, 60-cycle.
All transformers single-phase, 60-cycle.

Shop Equipment

	Price each
1 Rolin-Myer-Carpenter 18" x 12-ft. engine lathe. S.H...	450.00
1 Famous 36" band saw (Sidney Tool Co., No. 303), with tilting table	273.00
1 Oshkosh No. 1 rip and cross-cut saw, mounted	319.00
1 Blount No. 6 emery grinder, floor type, suitable for two emery wheels	47.00
1 Rockford 16" drill press; wheel and lever feed. S.H...	75.00
1 Royal steel forge; 10" fan, 22" hearth, 30" high	27.00
1 Canedy-Otto 12" pressure blower	15.00
1 Challenge power hack saw. S.H	35.00
1 Standard portable electrical drill, for holes up to 1¾", for 110-V A.C	115.00
1 Yale & Towne 2-ton "Triplex" chain hoist	91.00
1 American model "A" gasoline storage outfit, with measuring pump, 282-gal. tank	148.00
1 General Electric motor; 10-H.P., 220 volts	300.00

Miscellaneous Equipment

1 Wooden traveling stiff-leg derrick; mounted on trucks for 28-ft gage track, 30-ft. steel boom, 1-yd. clamshell bucket; operated by Thomas electric hoist, class "A," double drum, with type OX boom swinger; driven by 55-H.P. Wagner motor. S.H	3,818.00
1 American wooden stiff-leg derrick; 5-ton capacity, 40-ft. mast, 60-ft. boom, 12-ft. bull wheel; operated by American 8¼" x 10" double drum steam hoist engine and Lidgerwood reversible swing engine, size 154½, double cylinder 5" x 6", single drum, with 30-H.P. Clyde vertical boiler. S.H	4,031.00
2 Vulcan No. 2 steam pile drivers; 10½" cylinders, 36" stroke, 3000-lb. striking weight, 50-ft. leads of 8" x 10" pine.	
1 @	600.00
1 S.H. @	475.00
1 Ingersoll-Rand steam hammer, for driving 3" wood sheet piling. S.H	300.00
1 Worthington duplex steam pump; 10" x 6" x 10". S.H.	225.00
1 Cameron piston pump; 7" x 6" x 7", with 8-H.P. Gem City boiler	598.00
1 Worthington 6" centrifugal pump, class "C"; belt-driven with 30-H.P. Allis-Chalmers motor	498.00
1 Platt 4" centrifugal pump; 138½-ft. head, 1100 R.P.M., 550 G.P.M.; belted to 20-H.P. Westinghouse motor	383.00
1 American 3½" centrifugal pump, 120-ft. head, 1750 R.P.M., 450 G.P.M.; direct connected to 25-H.P. Wagner motor	435.00
1 Weinman centrifugal pump, 3" suction, 2" discharge, class "E", 80-ft. head at 1750 R.P.M., 35-ft. head at	

	Price each
1150 R.P.M.; direct-connected to 3-H.P. Allis-Chalmers motor	218.00
1 Dobbie hand winch; 4½" x 16" drums	54.00

The total cost of the equipment items for the Hamilton work was $248,400.00. This, as in the other cases, includes equipment items only, that is, such items as have an independent value and use when considered apart from the plant as a whole.

In contrast with this, the cost of *plant* at Hamilton including these equipment items, together with foundations, temporary railroads, temporary buildings, and other such constructions which go to make up a working plant was	$374,100.00
To this must be added the cost of installation, (setting up equipment, assembling machines, etc.)	28,500.00
And the cost of dismantling	2,800.00
Making the total plant investment	$405,400.00

All of which, however, was not necessarily on the job at one time, nor for the full construction period.

The total cost of work at Hamilton was $2,648,000, of which the total plant investment was 15%. This, it will be noted, is a much smaller percentage than the corresponding figure at any of the jobs mentioned heretofore. The reason is that the Hamilton work was more of a straight-away excavation job than any of the others, and for that reason less equipment was required compared with the value of the work done. Of this total investment the value of equipment items only, was 61%.

Troy Channel Improvement

Description. The work at Troy consisted in the enlargement of the channel through the city, the improvement of conditions below the city to permit free discharge of the flood flows, and the construction of a levee system particularly at the upper end of the city. The plan involved the lengthening of two highway bridges within the city limits, which developed into a complete reconstruction job at the Adams Street Bridge.

The improvement of the channel below the city consisted of a cutoff channel cutting off a long bend in the river, the replacement of an old levee to a more favorable location, and the clearing of trees and brush over a considerable area.

The principal quantities of work at Troy were as follows:

Excavation	330,000	cu. yds.
Levee embankments	167,000	" "
Concrete in slope revetment	1,500	" "
Concrete in walls and small structures	1,000	" "
Concrete in Adams Street Bridge	2,600	" "

The work was started after the other larger jobs were well under way, and the construction program was fixed so that the work would be completed by the time the dams were ready to function.

Excavation Plant. It turned out to be economical to contract most of the levee construction at Troy, so not much excavating equipment was required to be furnished by the District. On the contract for that portion of the river channel excavation which involved wet excavation, however, it was provided that the District would furnish the equipment, for which an agreed rental charge would be paid by the contractor. For this work a 60-ft. boom dragline machine, a pump, and a Bagley grader, were all that were required. The Bagley grader was used very successfully for cleaning out the material under the bridges and dragging it to a point where the dragline could pick it up.

Concrete Plant. The slope paving at Troy, as at the other places, was handled by contract, so no equipment was required for that work. For the river walls and minor structures a small mixer was all that was required. Most of the concrete plant required at Troy was for the reconstruction of the Adams St. Bridge.

The old bridge at Adams Street had a waterway which was entirely inadequate to match up with the capacity of the improved river channel The arches were not high enough to give the required clearance, the pier foundations were so shallow that they were not secure, particularly under improved channel conditions, and the length needed to be increased by 200 ft. Rather than wreck and remove the old bridge, and build a new one in its place, it was decided to make use of the old piers by underpinning them to the necessary depth, including a pile foundation, and by encasing them in a jacket of reinforced concrete; and to use the arches of the old bridge, before their removal, as supports for the new arches which would be built over the top of them. This permitted the old channel of the river to be kept clear of false work, and permitted the work on

the new bridge to proceed without danger of damage from floods. An account of the construction of this bridge will be found elsewhere. The principal items of equipment for this work were selected from that which had been in use on other jobs. The underpinning and pile driving at the old piers was handled by contract, so no District equipment was needed for that. For the main concrete work a portable gravel screening plant was required; two guy derricks, one for handling concrete stock, and another for handling forms, concrete, and other miscellaneous uses on the bridge itself; two Plymouth gasoline locomotives and some 1½-yd. V-shaped side dump cars for transporting concrete. An air compressor and a jack hammer drill, were needed for trimming up the old piers, and the ultimate removal of the old arches.

Shop Equipment. A small shop was established principally for use in connection with the Adams Street bridge work. The equipment of this shop is very similar to that listed under the other jobs.

Power for operating the electric equipment was secured from the local company, and taken off their regular distributing lines. The power consumption was comparatively small, not exceeding 60,000 K. W.-hours all told.

Table XXIV.—Equipment List—Troy Channel Improvement

Note: Items purchased second-hand are marked S. H. All others were new.

Excavating Equipment

		Price each
1	Bucyrus class 14 steam dragline excavator, mounted on caterpillars, 60-ft. boom, 2-yd bucket, main engine 8 x 10, swing engine 5½ x 6, boiler 54" x 9 ft., horizontal, total working weight 75 tons. S. H.	$23,000.00
1	Dean steam piston pump with 8-H. P. boiler. S. H.....	300.00
1	Bagley grader, with 1-yd. bucket, operated by a Mundy 8" x 12" double drum hoist, with 30-H.P. boiler.	
	Bucket .....$ 250.00	
	Hoist & Boiler. S. H. 1800.00	2,050.00

Concreting Equipment

1	Smith concrete mixer, size 28S, mounted on skids, power tilt, and batch feeding hopper, driven by direct connected 20-H. P. Allis-Chalmers motor	1,880.00
1	Jaeger concrete mixer, size ¼-yd., gasoline driven, power side loader, overturning drum	475.00

		Price each
2	Plymouth gasoline locomotives, model "AL," 3-ton, 36-in. gage	1,725.00
1	American wood stiff leg derrick, 40-ft. mast, 60-ft. boom, 12-ft. bull wheel. Operated by 55-H. P. Thomas double drum hoist with attached swinger	2,218.00
1	Bedford 9-ton steel guy derrick, 120-ft. mast, 105-ft. boom. Operated by 8 x 12 double drum Mundy hoist with 30-H. P. boiler and Lidgerwood size 154, 5 x 6 reversible swinger. S.H.	3,344.00
1	Western portable gravel screening plant, 25-yard, 4 compartments, with revolving screen, for ⅜", 1½" and 3" mesh, 28-ft. bucket elevator. Operated by 7-½-H.P. Allis-Chalmers 1200 R.P.M. motor	1,630.00
1	American 4" centrifugal pump direct connected to 25-H.P. Wagner motor	561.00
1	Snow steam pump with 10-H.P. Gem City boiler	375.00
4	Western 1-½-yd. V shaped steel side dump cars, 36" gage. S.H.	90.00
3000	ft.-30 pound relayer rail, total	450.00
1	Ingersol-Rand jack hammer drill, rotator type, style 100 BCR4	137.00
1	Sullivan air compressor, class WG3, size 10 x 10 single stage, capacity 213 cubic feet at 100 pounds, with 30" x 6 ft. vertical air receiver. Operated by belt connection to 50-H.P. Allis-Chalmers motor	1,438.00
	Shop Equipment	
1	Famous band saw, 36", style 303, with tilting table	173.00
1	Superior square base drill press, 20", wheel and lever feed	141.00
1	Blount No. 6 emery grinder, floor type	47.00
1	Royal steel forge, size 40	27.00
1	Buffalo blower, size 3	18.00
1	Oshkosh size No. 1 rip and cross-cut saw, mounted	319.00
1	Crescent planer and jointer, S.H.	160.00
1	Anvil	37.00
1	Allis-Chalmers 10-H.P. motor, 1150 R.P.M. type "AN"	272.50

The total value of these equipment items is $42,860. The *plant* cost, at Troy, was $55,600, to which was added $14,000 for installation and $3,200 for dismantling, making the total plant investment, $72,800. Of this amount, the cost of equipment items alone, was 59%. Only a part of the Troy work was done with this plant, as much of it was contracted.

Franklin Channel Improvement

Description. At Franklin, the channel improvement work consisted of two independent jobs, one on the east side of the

river and one on the west side. The work on the west side of the river was all let out by contract and consisted principally of levee construction from side borrow with a dragline machine.

On the east side of the river was one of the few places where 4-yard cars were loaded by dragline machine and used for levee construction. Here the work consisted in removing a projection of the east bank of the river and shaping up the slope, and the construction of a levee along the river front, tying into the high ground above and below the village. The estimated quantities were 95,000 cu. yds. of river bank excavation, and 70,000 cu. yds. of levee embankment. There was no available space for the direct disposal of the excavation without hauling, neither was there side borrow available for the levee work. For this reason it seemed logical to use hauling equipment, so that the levee might be built from the river bank excavation. A dragline machine was obviously the most economical equipment for the excavation work, and for the levee construction light hauling equipment was desirable.

Excavation Plant. A steam dragline machine, with 75-ft. boom, 1½-yd. bucket was available for the excavation work, and to go with this, thirty-six 4-yard, 36-in. gage dump cars, three 18-ton dinkey locomotives, and one 36-in. gage spreader car were secured. This combination was very satisfactory, and this same hauling equipment was used at another location where a rented dragline machine did the loading.

Concrete Plant. The concrete work at Franklin outside of the revetment work, which was handled by contract, consisted principally of a large retaining wall along the river bank, and the reinforcement and protection of a similar wall, which had been built by the County some time before. This work involved the placing of about 2,200 cu. yds. of concrete. Gravel was available in a pit not far away. A gravel bin and charging hopper were set up at one end of the wall where a ½-cu. yd. mixer was installed. A traveling stiff-leg derrick with 75-ft. boom was used for handling the concrete, which was carried from the mixer to the derrick by means of concrete buckets on trucks, horse-drawn over a narrow gage track. The base of the wall called for a sheet pile cutoff, which was driven by a small pile driving rig with a No. 2 steam hammer.

Shop Equipment. A small blacksmith shop was set up, as a matter of convenience and economy. As will be seen from the list of equipment no woodworking machinery was required here.

Table XXV.—Equipment List—Franklin Channel Improvement.

Note: Items purchased second hand are marked S.H. All others were new.

Excavating Equipment

		Price each
1	Bucyrus class 14 steam dragline excavator, skid mounted, 60-foot boom, with 15-foot extension, 1-½-yard bucket main engine 8 x 10, swing engine 5 x 6, boiler 54" x 9-½ ft., total working weight 75 tons. S.H.	$15,500.00
3	Porter 20-ton, saddle tank locomotives, 36" gage, type 040, 12 x 16 cylinders, 9200 pounds tractive force. S.H.	3,750.00
36	Western side dump cars, 4-yard capacity, wood bodies, 36" gage	300.00
8000	ft. 60 pound rail @ $45.00 per ton, total	3,200.00
1	Caldwell No. 104 standard water tank, 16-ft. diameter, 13 ft. 5" deep. Made of Louisiana cypress	220.00
1	Cameron 7 x 6 x 7 piston pump with 10-H.P. Gem City boiler. S.H.	550.00
1	Western spreader car, 36" gage, 18-ft. body, 15-½-ft. blades, equipped with single drum steam hoist engine. S.H.	1,000.00

Concreting Equipment

1	American 5-ton wood stiff leg derrick, 40-ft. mast, 60-ft. boom, 12-ft. bull wheel. Operated by an American 8¼" x 10" double drum steel hoist with Lidgerwood steam swing engine, double cylinder 5" x 6" with single drum, reversible type, 30 to 40-H.P. Clyde vertical boiler, S.H.	4,031.00
1	Smith concrete mixer, size 14S, on trucks. Power tilt, side loader, direct connected to 10-H.P. Smith steam engine and 12-H.P. boiler	2,027.00
1	Vulcan steam pile driver, size 2, 10½" cylinder, 36" stroke, 3000 pounds striking weight, 20-ft. leads. S.H.	475.00
5000	ft. 30-lb relayer rail, total	750.00

Shop Equipment

1	Superior 20" drill, square base, wheel and lever feed, stationary head	141.00
1	Royal No. 40 steel forge, 10" fan, 22" hearth	27.00
1	Buffalo No. 3 blower	18.00
1	Anvil	37.00
1	Allis-Chalmers 10-H.P. motor, 440 volts, 1150 R.P.M., type "AN"	272.50

The total cost of these equipment items is $50,600. The *plant* cost at Franklin was $55,900, to which were added $7,400 for

installation and $3,600 for dismantling, making the total plant investment $66,900. The difference here between the equipment cost and the total plant investment is less than at any of the other jobs mentioned, due to the type of equipment listed, a large percentage of which consists of standard excavating equipment on which the cost of installation and additional plant required is less than for most other types of construction equipment. Only a part of the work at Franklin was done with this equipment, as a considerable portion of that work was done by contract.

General Repair Shop

Description. The General Repair Shop, which was maintained at Dayton, has been referred to in Chapter 1. It is thought that a list of the equipment for that shop, included here, will be valuable to those who may have need for a similar installation.

Here was a shop that took care of most general repairs, and many emergency repairs, for the eight or ten large construction jobs being carried on around it, at distances varying from two or three to twenty or thirty miles. As has been explained before, a small shop was maintained at each job, where minor repairs, and such emergency repairs as could be handled by the local forces, were taken care of. With truck service between Dayton and all these jobs, however, and a railroad connection right into the shop building, it was feasible to bring into the general repair shop many items of repairs which otherwise could not have been so handled. A large percentage of the work consisted of rush work where speed was the important item, but there was always a quantity of routine work on hand to keep the force busy at other times.

When the time came that some of the jobs were being finished, and some of the equipment being released, most of such equipment was overhauled and put in shape for sale by the shop forces. As far as practicable the released equipment was shipped in to the shop for that purpose, although in some cases a crew from the shop went out into the field to do the work. At that time a large equipment yard was established on some vacant land across the street from the shop, and the railroad connections were extended over into the yard. As the work on the jobs began to let up, the work of overhauling released equipment increased, until finally the latter became the principal work of the shop forces.

To give an idea of the amount of work which was handled by this General Repair Shop, the total value of shop orders handled during one calendar year was over $330,000.00.

Table XXVI—Equipment List—General Repair Shop

Note: Items purchased second-hand are marked S.H. All others were new.

	Price each
1 Locomotive Crane, assembled, 10 to 15 tons capacity 25-ft. boom, 9 x 10 main engines—35-H.P. Vertical boiler S.H.	$ 6,500.00
1 Worthington Laidlaw air compressor—9-½" x 12" single stage, with 36" x 84" air receiver, 100-lb. working pressure, belted to a 50-H.P. Allis-Chalmers cage type motor	1,570.00
1 LeBlond 18" x 14-ft. quick change lathe—double friction, back gear taper attachment, 15" jaw	1,760.00
1 Pond 42" engine lathe—S.H.	2,600.00
1 Queen City 20" shaper—S.H.	600.00
5 K & G Acetyline cutting and welding outfits	698.00
1 Brown No. 4 bolt cutter—on skids, with taps and dies for ⅜" to 2-½" pipe and bolts, S.H.	400.00
1 Fitchburg 24" x 12-ft. lathe, S.H.	750.00
1 Rockford 18" x 18-ft. lathe, double back geared, S.H.	550.00
1 LeBlond 18" x 10-ft. quick change lathe—3-step cone, double friction, back geared, S.H.	900.00
1 Bement air hammer—800-lb. single frame	1,800.00
1 Canedy-Otto No. 101 radial arm drill press, 3½-ft. arm, floor type, power feed, S.H.	500.00
1 Rockford 16" drill press floor type, wheel and lever feed, S.H.	75.00
1 Silbey-Ware 24" drill press, floor type, wheel and lever feed, S.H.	225.00
1 Cincinnati 24" drill press, floor type, wheel and lever feed, S.H.	100.00
1 Punch and shear machine, S.H.	475.00
1 Forbes pipe cutter and threader—with guides and dies 2-½" to 8", S.H.	400.00
1 Hawes and Phillips 6-ft. planer, S.H.	250.00
1 South Bend 10" screw cutting lathe, S.H.	175.00
1 Atkins power hack saw, S.H.	175.00
1 Sterling No. 1 power hack saw, S.H.	35.00
1 Tate-Jones oil burning rivet forge	122.00
2 Thor close corner type air drills	150.00
1 Ingersoll-Rand air drill—Size W-9, Little David 3" hole, 2" tap	160.00
2 Heis-Wolfe electric drills ½" hole, 110-V., model 20 S.H.	100.00
1 Van Dorn electric drill ½" hole, 110-Volt	110.00

		Price each
1	Cincinnati electric emery grinder—type SSA. Wheel and arbor for inside grinding	96.00
2	Royal portable steel forges—16" fan, 22" hearth	27.00
2	Green air riveting hammers—model 90	50.00
3	Yale-Towne 2-ton triplex chain hoist—9-ft. list, 11-ft. reach, 120-lb. pull at full load	91.00
1	Buffalo blower Size No. 4	55.00
1	Buffalo blower Size No. 2	18.00
19	Joyce-Cridland jacks, six 50-ton, five 25-ton, four 35-ton, four 75-ton, total	1,992.00
1	Famous rip and cross cut table saw, S.H.	175.00
1	Gentel-Margedant 16" power jointer, S.H.	150.00
1	Wood turning lathe, S.H.	60.00
1	Silver 36" band saw, S. H.	235.00

The total cost of these equipment items was $27,430.

Central Garage

Description. The Central garage and repair shop for motor vehicles has been mentioned heretofore. This garage was located at Dayton, adjacent to the general repair shop and the general warehouse. The repair and upkeep of all motor vehicles owned by the District was handled by this garage, and with a few exceptions all the vehicles were kept there and routed out as needed. One man gave practically his entire time to the dispatching of trucks during the busy construction period. In a few cases a truck was assigned to a certain job exclusively, and kept there for long periods, and it was the general practice to assign one or more of the Ford light-delivery cars permanently to each of the larger jobs, for use of the superintendent, general foreman or master mechanic. Most of the Dodge touring cars were assigned permanently to division engineers, although two or three of them were kept at the garage for general use. In other cases, whenever it seemed desirable, some of the other touring cars were assigned for the exclusive use of one person during a certain period, but wherever feasible it was found to be much more satisfactory to make a definite arrangement with an employee who needed a car for official use, to use his own car for a certain agreed amount per month. This was less expensive for the District, more satisfactory to the employee, required fewer cars to be owned by the District and allowed more flexible use of those which it did own.

Following is a complete list of the motor vehicles owned by the District, together with the more important items of general equipment at the central garage.

Table XXVII—Garage Equipment

Items purchased second-hand are marked S.H. All others were new.

	Price each
5 Pierce-Arrow 5-ton trucks, 14-ft. wheel base, steel dump bodies	$5,425.00
1 Pierce-Arrow 5-ton truck, 17-ft. wheel base, stake body with Mead-Morrison power winch	5,302.00
7 Pierce-Arrow 2-ton trucks, 15-ft. wheel base, stake body	3,277.50
1 Reo one-ton truck. Model F—express body with canopy (assigned to Central Commissary)	1,710.00
1 Reo one-ton truck. Model F—8-ft. closed panel body (assigned to Central Commissary)	1,625.00
1 Reo one-ton truck. Model F—standard open truck body	1,425.00
9 Ford one-ton trucks, Model T with No. 603 Parry truck body	773.33
9 Ford delivery trucks. Model T with Parry light delivery body	515.50
6 Fords Model T. chassis with No. 404 Parry depot wagon bodies	552.43
1 Cadillac, Model 53, 7-passenger touring car, S.H.	950.00
1 Hudson, Model 6-40, seven passenger touring car, S.H.	433.00
1 Studebaker Roadster, S.H.	500.00
10 Dodge Bros. standard touring cars—	
9 @	885.00
1 @	1,160.00
12 Ford Model T touring cars	585.00
8 Ford Model T roadsters with No. 208 Parry slip-on-body	417.57
7 Ford Model T runabouts	555.00
1 American gasoline measuring pump Model A, 10-gallon register with 1000-gallon tank, S.H.	248.00
1 American portable gasoline tank and pump, 50-gallon capacity	180.00
1 Miller No. 17, three cavity steam vulcanizer gas burner	130.00
1 General Electric Tungar Rectifier, 3 to 30-cell size	124.00
1 Chisholm-Moore 3-ton chain hoist with traveler	218.00
1 Jackson Electric valve grinder, size 4	36.00
1 Heisey-Wolf electric drill up to ½" holes	100.00
1 Davis size 8 auto curtain sewing machine	47.00

Central Commissary

Description. At each of the five dams it was necessary to establish and maintain a camp to house the workmen, and at

each camp there was a general store, and a mess house to accommodate several hundred men. At Taylorsville Dam, which was somewhat centrally located as regards the others, a bake shop, meat shop, refrigeration plant and meat storage were provided, from which the other camps were supplied.

The cold storage for meats had a capacity of about a carload, with facilities for quick handling in and out of storage. A meat shop was maintained in connection, where meat was cut, trimmed and boned before being delivered to the various mess houses. Meat was also prepared at this shop for family trade, was delivered to the various camps on advance orders, and sold through the stores. Thus all the trimmings, bones and waste

FIG. 37—5-TON TRUCK AND TRAILER

were accumulated at the one place, and were there turned into by-products (lard, ground bone, etc.), while if this waste had been scattered out at five camps there would not have been enough of it at any one place worth bothering with. As an example of how this worked out,—all the lard used at the bake shop and the five mess houses was a product of this central commissary, and in addition to that, a great deal was sold to the families at the camps.

The bake shop had a capacity of about three hundred loaves of bread and one hundred pies per day. In addition to the demands of the five mess houses, the family trade took a considerable amount of the output of the bake shop.

The refrigerating machine was rated as a six-ton plant, which means that its output in refrigeration was equivalent to the melting of six tons of ice per day. This machine was capable of

supplying the refrigeration needed for meat, egg, butter and lard storage, storage of baked goods, and in addition it had the capacity to manufacture about two tons of ice per twenty-four hours. This ice was used at Taylorsville, and the two camps nearest, and the supply was sufficient except during extremely hot weather, when it was necessary to purchase locally to take care of the demand. Another small ice machine was installed at the Germantown camp; and Lockington, where the force was smaller than on the other jobs, was supplied locally.

Two delivery trucks were kept busy distributing the supplies from this central commissary, one being on the road every day, and the other held in reserve to take care of special trips and peak demands. During the busy seasons the commissary truck averaged more than a hundred miles a day on the road.

It is thought worth while to include here a list of the principal items of equipment for this central commissary, and also, under separate heading, the equipment for one of the mess houses, which is typical of the others. This mess house was equipped to handle a force of about 350 men, working two 10-hr. shifts.

Table XXVIII—Equipment List—Central Commissary

Note: Items purchased second-hand are marked S.H. All others were new.

	Price each
1 York 6-ton refrigerating plant—with compressor, tanks, etc., operated by 10-H.P. General Electric motor and 2-H.P. Century motor	5,050.00
1 Rinck & Bros. bake oven, malleable front, body of brick construction. Capacity 400 loaves	1,526.00
1 Schmidt No. 1 refrigerator—21 ft. long, 16 ft. wide, 12 ft. high, brine coils, for meat storage	1,524.00
1 Schmidt No. 2 refrigerator—16 ft. long, 7 ft. wide, 11 ft. high with brine coils, for bakery storage	634.00
1 Van Range "Century" electric dough mixer, single blade, 2-bbl. capacity	431.00
1 Tyson ice cream freezer—model "C" power driven	325.00
1 Buffalo electric meat chopper—18" x 20" bowl	240.00
2 Van Range 60-gal. cast iron jacketed steam kettles, brass fitted	145.00
3 Van Range galvanized proof ovens, equipped with steam appliances, 7 ft. high, 6 ft. long, 30" wide, 7 shelves	120.00
1 Wharton power ice breaker "Creasey" brand, model 52Y	75.00

	Price each
2 Van Range Sanitary dough troughs 3/16 boiler plate, mounted on wheels	59.00
4 Van Range vegetable steamers, 2-bushel capacity, heavy cast iron	48.00
2 Van Range bakers tables. Maple strip top, iron legs	45.00
2 Buckeye bread racks	41.00
1 Van Range pastry stove	37.50
1 Van Range bakers sink 24" x 24" x 12" with drain boards	35.00
1 Van Range portable pan rack, 34-pan size	28.50
1 Van Range lard press. Hand power	28.00
1 Mann green bone grinder	80.00
2 Reo one-ton auto trucks, pneumatic tires.	
One with standard 8-ft. panel body	1,625.00
One with standard express body, canopy top	1,425.00

Table XXIX.—Equipment List—Typical Mess Hall and Kitchen

Note: Items purchased second-hand are marked S.H. All others were new.

	Price each
1 Van Range kitchen range—2 fire, 2 oven, style No. 405	$ 248.00
1 Dalton-Whittier-True 18-ft. x 24" copper service counter and steam table	445.00
1 Crescent electric dish and glass washer—Model "AA" with baskets	325.00
1 Schmidt No. 3 refrigerator—12 ft. long, 3 ft. wide, 9 ft. high, ice cooled	409.00
1 Dalton-Whittier-True 10-gal. steam coffee urn with 20-gal. hot water tank—copper and nickel plated	140.00
1 Van Range dish table L shape, maple and galvanized iron 16 ft. outside perimeter. Used with dishwasher	80.00
1 Cooks table, 24" x 9-ft. maple strip top, iron legs. With Cooks sink at end, 12" x 24" x 12" heavily galvanized	66.00
1 Van Range sink, heavy galvanized iron 48" x 24" x 16" with 2 24" drain boards	54.00
1 Van Range sanitary sink, 48" x 24" x 16" heavy galvanized iron, 2 compartments, splash board and 2 30" drain boards	54.00
1 Van Range saucepan rack 5 ft. long made up of bar iron	23.00

CHAPTER IV.—EXCAVATION METHODS AND COSTS

Earth and Rock Excavation for Outlet Structures at Dams

The outlet excavation at the dams consisted in each case of both earth and rock, part of which was below water level. In every case except one, the earth and rock were taken out together by the same equipment, as one operation. At Taylorsville, however, the rock excavation was a separate job. These jobs may be divided roughly into two principal groups, the first including Germantown and Englewood, where the outlets consisted of twin conduits; the second including Taylorsville and Huffman, where the combination outlet and spillway structure was used. The excavation at Lockington was largely an open gravel cut, with a little rock excavation at the bottom, and was of such nature that a discussion of it would be of no special interest under this heading. A brief description of each of these jobs, together with the reasons for selecting the excavation plant for each, has been given in Chapter III.

Outlet Excavation at Germantown and Englewood Dams. The outlet excavation at Germantown consisted principally of a trench about 30 ft. wide and 550 ft. long, with an average depth of about 28 ft., the lower half or more of which was in rock. This rock was what is known as Cincinnatian formation, and consists of layers of hard limestone from a fraction of an inch to several inches in thickness interstratified with hard clay or shale. The shale remains hard and firm, where protected from the action of the elements, but slacks rapidly on exposure. At the downstream end the excavation was widened out to about 100 ft., and also was carried about 10 ft. deeper into the rock, in order to accommodate the stilling pool. At the upstream end the excavation was also widened to take care of the bell-mouth inlet.

The conduits were located along the foot of the hill, some distance away from the river channel. The material overlying the rock consisted of clay and gravel, and varied in depth generally from 12 to 16 ft., although it was considerably more in places on

the up-hill side. The proportion of rock in the excavation, according to final figures was about 40%.

The work was started at the downstream end, after building a cofferdam to protect against flooding from the river. The material was handled by a Lidgerwood electric dragline machine with 100-ft. boom and 2½-yd. bucket, working from the top of the cut and cleaning up the excavation ready for concrete, as it moved upstream. The earth overburden was thrown over into the dam section, where it was used later as a part of the embankment, being worked into place by sluicing where necessary.

As soon as a 40 or 50-ft. stretch of rock had been laid bare, steam tripod drills were set up and the rock was drilled to within a few inches of the bottom grade. Generally speaking, the holes were drilled on about 5-ft. spacing, and were loaded with about a 10-lb. charge of 40% dynamite. The drilling and shooting was usually done by the night shift, so that the dragline machine had the material to work on during the day shift. Special care was taken to avoid overbreakage, as every yard taken out beyond neat lines meant also an extra yard of concrete. This was very difficult, however, as the rock broke out so irregularly, and the only way to avoid excessive overbreakage was to keep the drill holes well within neat lines. This resulted necessarily in a large amount of trimming, which was taken up as a separate operation, but followed closely on the heels of the rough excavation, so that the material could still be handled by the dragline machine. This trimming was done with jap drills, picks and bars, removing all material that had been loosened by the shooting or other operations.

The rock excavation was loaded into 12-yard standard gage dump cars and hauled to the spoil bank about half a mile away. The trains were made up of four or five cars each, and were hauled by 40-ton saddle tank standard gage locomotives. As soon as a sufficient amount of excavation had been completed, the concrete work was started, thus permitting the two jobs to proceed simultaneously.

Very little trouble was encountered on account of water. One 4-in. pump, operated at infrequent intervals, was sufficient to take care of all seepage. This was due largely to the care with which the cofferdam was built.

A fair month's work was from 9,000 to 12,000 cu .yds., while the best month showed an output of over 16,000 cu. yds., the

dragline working one 10-hour shift per day. This was difficult work for any type of equipment, as is indicated by a glance at Fig. 38. The long, deep, and comparatively narrow trench, and the large amount of trimming as compared with the total excavation, meant slow progress at best. The equipment and methods adopted seemed to be well suited to the job.

FIG. 38—ROCK EXCAVATION FOR OUTLET CONDUITS AT ENGLEWOOD DAM

This illustrates a narrow trench which it was necessary to dig for the outlet conduits at Englewood and Germantown. The dragline machine could dig close to neat lines and clean up the trench as it moved along. The concrete work could start as soon as the excavation was well under way.

At Englewood the job was very similar to the one at Germantown. The conduit trench was somewhat longer, being about 700 ft. long at Englewood, and the earth overburden was considerably less, averaging not more than 2 ft. The top 6 to 10 ft. of Cincinnatian, however, was soft enough so that it could be excavated by the dragline machine without shooting. Therefore

the same general method of procedure was employed as at Germantown; i. e., to remove the overburden and softer material on top of the hard rock, leaving a bench of rock to be drilled and shot,—usually by the night shift. The excavation, including the trimming, was completed as the dragline moved on, except that during a part of the time the trimming was cleaned up by

FIG. 39—ROCK EXCAVATION FOR STILLING POOL AT GERMANTOWN DAM

This shows the pit where the excavation was widened out to make room for the stilling pool. Even in the widest of these excavations the dragline machine could complete full width as it moved along so that concreting could follow without delay.

a locomotive crane, which was available on part time in connection with other work. The final classification of material at Englewood showed the proportion of rock to be about 75%.

The excavating machine here was a Bucyrus steam dragline with an 85-ft. boom and a 5-yd. bucket. All of the material, including the overburden, was loaded into 12-yd. standard gage dump cars, and hauled away in four or five-car trains by means of 40-ton saddle tank standard gage locomotives. The distance to the spoil banks was less than at Germantown, averaging not

more than a quarter of a mile. The progress of work was about the same as at Germantown, that is about 12,000 cu. yds. for an average month, with a maximum running somewhat over 16,000 cu. yds. per month. Fig. 39 is another view of this class of excavation, where the trench is widened out to make room for the stilling pool.

Below are given cost records for this excavation. These figures include the cost of trimming. At Germantown 98,000 cu. yds., consisting of about 60% earth and 40% rock, were moved at a unit cost of $1.29 per cu. yd. At Englewood 67,000 cu. yds., of which about 75% was rock, were moved for $1.60 per cu. yd. Most of the difference was in the labor item, and is accounted for by the higher percentage of rock in the material handled.

These figures include general superintendence, full plant depreciation, and all costs properly chargeable to the work. The cost of headquarters office at Dayton was carried as a general expense item and was not distributed. This work was done at a time when labor was being paid 35c to 40c per hour, dragline operators $175 per month, blacksmiths 62½c per hour, locomotive runners 50c per hour, and drill runners 50c per hour.

All men on an hourly basis were given 11 hours' pay for 10 hours' work. (10-hr. shifts with time and a half after 8 hrs.)

Cost of Excavation for Outlet Works at Germantown and Englewood Dams

Germantown—98,000 cu. yd. {60% earth, 40% rock}

		Unit Cost	Percent of Total
Superintendence and Overhead..	$ 6,690.48	$0.068	5.3
Labor	63,265.16	.646	50.0
Supplies	34,973.33	.357	27.7
Plant Depreciation	21,418.25	.219	17.0
	$126,349.21	$1.290 per cu. yd.	

Englewood—67,000 cu. yd. {25% earth, 75% rock}

		Unit Cost	Percent of Total
Superintendence and Overhead..	$ 5,254.80	$0.078	4.9
Labor	58,563.86	.875	54.5
Supplies	26,240.35	.391	24.4
Plant Depreciation	17,409.15	.26	16.2
	$107,468.16	$1.604 per cu. yd.	

Outlet Excavation at Taylorsville and Huffman Dams. The excavation at Taylorsville was a broad deep cut, forming a bench into the side hill which came out to the river's edge at that point. The cut varied in width from about 150 to 260 ft., and was about 1,600 ft. long including the inlet and outlet channels to the old river bed. The deepest excavation was 40 ft. below water level in the river and more than 60 ft. below the top of the rock.

A total of about 700,000 cu. yds. of material had to be moved, about 500,000 cu. yds. of which was earth, suitable for use in the dam. The remainder was Cincinnatian formation, very similar to that at Germantown, except that some of the limestone layers were as much as eight or ten inches thick. The shale was very hard. Because of the deep side hill cut the structure was located as far out towards the river as was possible, without sacrificing a good rock foundation for the whole structure. This necessitated a cofferdam along the river's edge for a length of about 500 ft., which was built of earth from the excavation, with a puddled cut-off down to the rock surface.

The large quantity of earth overlying the rock at this site was moved mostly by hydraulic sluicing, and became a part of the hydraulic fill embankment in the dam. That was an entirely separate operation from the rest of the excavation, preceding the rock work, and will be described under another heading. The present discussion therefore will be confined to the rock excavation.

The work started at the downstream end and was completed to full width and depth as the excavating equipment moved on upstream. First, the rock was drilled by Sanderson "Cyclone" electric well drills, the holes being about 4½ in. diameter, spaced about 10 ft. apart, both ways. Fifty feet of hole per 10-hr. shift was considered a good day's work. The first holes were loaded with about 1½ lb. of 40% dynamite per cu. yd. of rock to be lifted, but after a start had been made and the work shaped up with a vertical face, the amount of explosive per cu. yd. was reduced to about ½ lb. The average for the whole job was 0.6 lb. per cu. yd.

For this excavation a Lidgerwood electric dragline machine was used, with 100-ft. boom and 3½-yd. bucket. The material was loaded into 12-yd. standard gage dump cars and hauled away

in 4 or 5-car trains by 40-ton saddle-tank standard gage locomotives. The distance to the spoil banks was about 1,000 ft.

This excavation was so wide and so deep, especially at the downstream end, that it was impracticable for the dragline to reach all the way across the cut from one position. It, therefore, was turned so that it moved back and forth across the cut at right angles to the center line of the excavation, or parallel to the face of the cut. This machine was mounted on trucks, and the movement upstream was accomplished by curving the track upstream slightly each time across the cut, and straightening up at the end so as to reverse the curve going back. The working face was wide enough so that the drills could be working on one side while the dragline was cleaning up on the other. After the work was well opened up it was found that shooting off the whole face at once, (a depth of 50 to 60 ft.) threw the top material so far out that it was difficult for the dragline machine to reach it. To avoid this, and to better break up the top layers, it was decided to drill and shoot in two lifts. Two rows of holes were drilled, 25 to 30 ft. deep. These were shot and the material cleaned up with the dragline. Then the drills were picked up by the dragline machine and set down on this bench, from which the bottom material was drilled and shot; but before this bottom material was cleaned up, two more rows of holes were drilled and shot down from the top. This method was much more satisfactory than the original one of shooting off the whole vertical face at one time. Fig. 40 is a view of this pit and some of the equipment used.

Some of this material near the bottom of the cut was so hard and firm that it broke up in chunks from 3 to 5 ft. in thickness. There were two or more well defined strata where this occurred, and some of the material had to be block-holed and broken up in the pit before it could be handled by the dragline. The material in these hard strata was no different than the rest, except that the layers of limestone and shale were so firmly held together that it had to be handled in the pit just as if it were a layer of solid rock.

In the drilling and shooting care was taken not to get any of the holes within 3 to 5 ft. of the neat lines on the sides, nor within 2 ft. of the finished grade at the bottom. Otherwise, there would have been danger of shattering or loosening the

rock outside the lines. This meant, of course, that the trimming was much more of a job than it had been at Germantown or Englewood; that is to say, it was heavier work, and much more material had to be moved. This trimming was handled, however, in practically the same way as at the other places; i. e., by the use of jap drills, picks and bars, and it was possible always to keep close up with the rough excavation so that most of the material could be handled by the dragline. Some difficulty was

FIG. 40—ROCK EXCAVATION AT TAYLORSVILLE DAM

The rock excavation at this point was over 200 feet wide and more than 60 feet deep. Nearly 200,000 cubic yards of rock were moved by the dragline, loaded into cars and hauled away. This view shows the wagon drills drilling for the second lift.

experienced in keeping above grade of the bottom excavation, because the constant dragging of the bucket over the bottom tended to continually loosen up the material. This was especially true at changes in grade where the bucket had to be dragged up an incline. At such places there was always a tendency for the bucket teeth to catch under a limestone layer and tear it loose. In this kind of rock it was much easier to hold to the lines when digging *down* an incline than when digging *up* the same grade.

An average month's work in this rock was about 15,000 cu. yds., and the best month shows an output of over 22,000 cu. yds. The work was carried on by two 10-hr. shifts per day, and because of the width of the cut, and the nature of the work, the drilling and blasting did not interfere to any great extent with the continuous operation of the dragline machine.

This was a rather unusual job for a dragline excavator, both because of the character of material handled, and because of the great depth of cut, which at one place was 63 ft. below where the machine stood. It was slow work at best. The wear on the drag cable was very severe and required careful watching. Frequent replacement of cable was necessary. The cost of cable alone ran as high as 3c per cu. yd. The wear and tear on the bucket was also a considerable item of expense. But the great advantage of having the equipment up on top of the cut, out of the way of floods and water troubles, and with easy grades to the waste banks, justified the use of the dragline outfit. It is doubtful if any other equipment could have handled the work as satisfactorily.

The job at Huffman was very similar to the one just described, except that the dimensions and the quantities were not quite as large. This cut also took the form of a bench into the bluff along the edge of the river. The hillside slope was not steep, as at Taylorsville. The width of the cut varied from about 120 to 210 ft., and its total length was about 1,400 ft. including the inlet and outlet channels to the old river bed. The material was very similar to that at Taylorsville, with some of the limestone layers as much as 15 in. in thickness. The total excavation amounted to about 175,000 cu. yds., of which about 80% was classified as rock.

The overburden and soft rock at Huffman had an average depth of about 10 ft. This could have been removed without blasting, but the starting of the work here was dependent on the relocation of a railroad, which ran about along the center line of the location for the outlet structure. While a temporary relocation of this railroad was being made, the drilling was started on one side of the old railroad cut. The first lift was drilled to a depth of about 20 ft., which included the soft rock and the overburden, so that just as soon as the traffic had been turned from the old line of the railroad these holes were loaded and shot and actual excavation for the outlet structure was begun.

The excavating equipment used here was a Bucyrus electric dragline machine with 100-ft. boom and 3½-yd. bucket. The material was loaded into 12-yd. standard gage dump cars, and hauled away by means of 40-ton standard gage saddle-tank locomotives, as at the other dams. All of this waste material was used in the embankment for the permanent railroad relocation. The average haul to this embankment was about half a mile.

The economical procedure at Huffman was to remove the material in two lifts, taking first that which had been drilled previous to the railroad relocation, and on the uphill side of the old railroad cut. The machine started at the upstream end of the cut and the abandoned railroad line was used as a loading track. As soon as that lift had been completed the machine tracked back, at the same time cleaning up on the other side of the railroad cut all material that could be removed without blasting. This brought the machine back to the upstream end of the work, ready to start on the straight rock excavation.

In drilling for the second lift, holes about 18 ft. deep were sufficient for the portion of the excavation above the stilling pool. Most of the drilling was done with "Cyclone" well drills similar to those used at Taylorsville, except that they were operated by steam, but at Huffman some drilling was done with an Ingersoll-Rand wagon drill. The wagon drill made much faster progress on actual drilling than did the well drill, but it was so much heavier and so much harder to move over the rough, irregular ground, that the actual cost of drilling by the two types of equipment turned out to be practically the same; i. e., about 31c per ft. A fair day's work for the well drill was about 45 ft. of holes per shift, while the wagon drill actually made one record of 240 ft. of holes in one 10-hr. shift. This, of course, was in a place where very little moving was required, and conditions for moving were favorable.

In shooting the holes at Huffman it was found advantageous to use a small springing charge before the final loading of the hole. Springing the holes, however, had to be done very carefully, because if an overcharge was used it tended to break off pieces of the limestone layers and plug the hole. The holes were put down on about 10-ft. centers, and it was the practice to load about 20 to 25 of these holes and shoot them simultaneously. The amount of explosives used on this work was about ½ lb.

per cu. yd. of material moved. The cost of drilling and blasting was 26½c per cu. yd.

When the deeper section of excavation was reached, another lift of holes was necessary to reach bottom grade. The cut was wide enough here, however, so that drilling and blasting could proceed at one side of the cut while the excavation was being made on the other; thus the entire cut was completed as the dragline passed through, as was done on the other jobs.

General excavation methods, and the trimming, were handled at Huffman in about the same way as at Taylorsville, except as has been noted. The fact that the Huffman excavation proceeded downstream, meant that the machine was excavating down the incline into the stilling pool instead of against the grade as at Taylorsville. This made it much easier to avoid tearing up the material from below grade, and emphasized what has already been mentioned, that in stratified material of this kind it is much easier, with a dragline machine, to keep close to the lines when excavating down an incline, than when going in the other direction.

In spite of the fact that a cofferdam was required all along the river side of the excavation, there was very little seepage into the pit except at high stages of the river, when a 4-in. and a 6-in. pump were required to keep the pit dry.

A rather unusual record was made at Huffman during the progress of this excavation, when the dragline machine took out a total of 25,000 cu. yds. of material while standing on the same spot. This was at a point near the downstream end of the stilling pool excavation where the bottom grade was nearly 60 ft. below the machine, and where the width of the cut was something over 210 ft. Fig. 17 (Chapter II.) shows this interesting feature of that work.

A good month's work at Huffman was about 20,000 cu. yds., and on two different months this ran up to nearly 25,000 cu. yds. per month. It happened that during both the months when these higher records were made the material handled was more than 90% rock. The work was carried on in two 10-hr. shifts per day.

Wear and tear on the dragline buckets has been mentioned as a considerable item of cost in connection with work of this character. A record was kept at Huffman which showed that

the average cost of bucket repairs on this heavy excavation amounted to 1½c per cu. yd.

Below are given cost figures for the excavation at Taylorsville and Huffman, including the cost of trimming. At Taylorsville 178,000 cu. yds. of rock were moved, at an average cost of $1.72 per cu. yd. At Huffman 173,000 cu. yds., of which 80% was rock, were moved at an average cost of $1.56 per cu. yd. These figures are made up on the same basis as those given earlier in this chapter for the work at Germantown and Englewood. They include general superintendence, full plant depreciation and all field costs. The wage scale was the same as that given for the other jobs; i. e., labor 35c to 40c per hour, dragline runners $175 per month, locomotive runners 50c per hour, drill runners 50c per hour, blacksmiths 60c per hour, all men on an hourly basis being given time and a half after 8 hrs., or eleven hours' pay for ten hours' work.

Cost of Excavation for Outlet Works at Taylorsville and Huffman

Taylorsville—177,800 cu. yds., all rock

		Unit Cost	Percent. of Total
Superintendence and Overhead..	$ 12,937.72	$0.073	4.2
Labor	152,899.12	.859	50.0
Supplies	103,622.05	.583	33.9
Plant Depreciation	36,333.61	.205	11.9
	$305,792.50	$1.72 per cu. yd.	

Huffman—173,000 cu. yds. { 20% earth, 80% rock }

		Unit Cost	Percent. of Total
Superintendence and Overhead..	$ 11,677.15	$0.067	4.3
Labor	151,583.91	.876	56.1
Supplies	81,471.51	.470	30.2
Plant Depreciation	25,341.55	.147	9.4
	$270,074.12	$1.56 per cu. yd.	

Earth and Gravel Excavation from Borrow Pits at Dams

More than five million cu. yds. of earth and gravel were excavated by dragline machines from borrow pits in the river valleys at Germantown, Englewood and Huffman Dams. All of this material was loaded into 12-yd. standard gage dump cars

and hauled to dredge pumps for use in the hydraulic fill embankment. At all three places a large proportion of the material was excavated from below water level. As far as affecting excavation methods or progress is concerned, the material was practically the same at all three places. The equipment was similar, and methods were standardized except as to slight

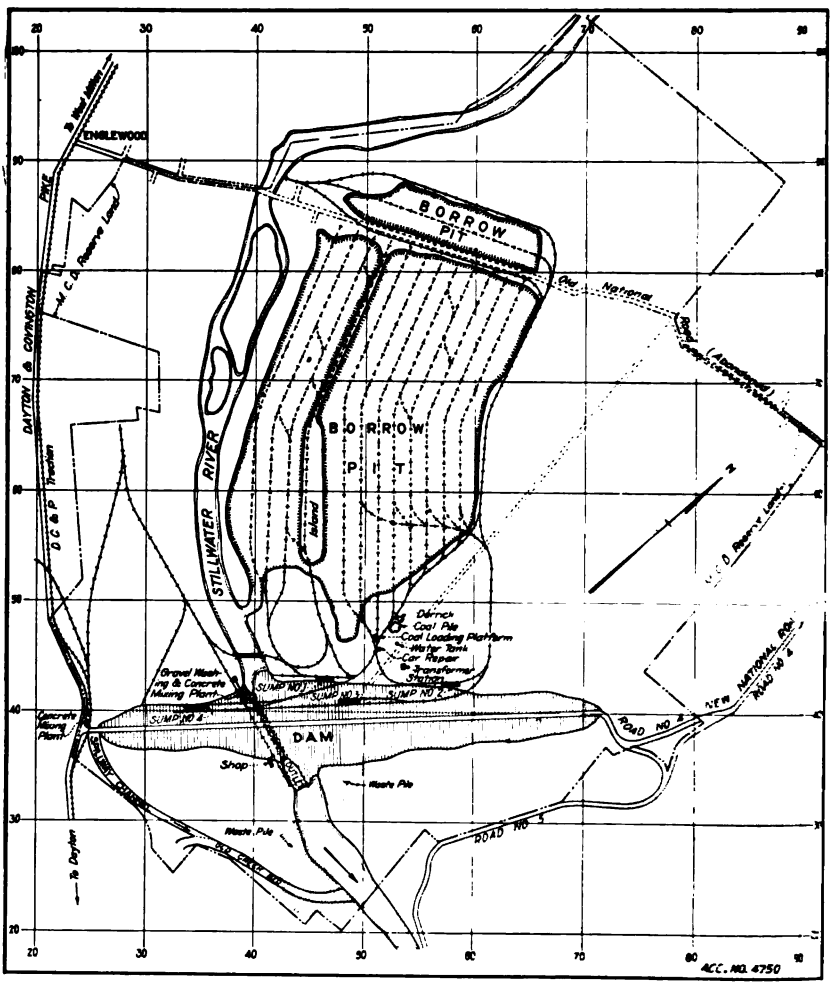


FIG. 41—BORROW PIT AT ENGLEWOOD DAM

This shows the layout of the borrow pit and track systems with reference to the dam itself. The various locations of the dredge pump sumps are shown also. Nearly 3,500,000 cubic yards of material were taken from this borrow pit for the hydraulic fill embankment.

modifications to fit local conditions. The largest quantity of material handled at any one place was at Englewood, and because of the size of the job, duplicate equipment was used there, as compared with single units at Germantown and Huffman.

Borrow Pit Excavation at Englewood Dam. The borrow pit at Englewood lay upstream (north) from the dam, east of the river, and extended northerly a distance of about three-quarters of a mile. (See Fig. 41.) Its width was about half a mile and was fairly uniform. The material was mostly river gravel and sand overlaid with 2 or 3 ft. of clayey loam, but the easterly side of the pit approaching the hillside was mostly a glacial till with the sub-stratum of gravel either very much reduced in thickness or entirely absent. While this till was excellent for furnishing the fines in the impervious core of the hydraulic fill, it contained such a large proportion of extremely fine particles that its use in the dam was limited. The depth of cut in the borrow pit varied from 15 to 20 ft., of which the lower 3 to 5 ft. was below water level. Nearly 3,500,000 cu. yds. of earth and gravel were taken from this borrow pit.

The excavation was performed by three dragline excavators; a Bucyrus steam machine, with 85-ft. boom, and 4½-yd. bucket; another Bucyrus machine exactly like the first, except that it was operated by electric power instead of by steam, and a Lidgerwood steam machine with 75-ft. boom and a 1½-yd. bucket. The latter was not used regularly for furnishing material to the dam, but was used for supplying concrete stock and various other purposes, and was called upon, if necessary, to supplement the output of the larger machines. Fig. 42 is a general view of the borrow pit, and some of the equipment that was used on this work. The cuts ran north and south in the pit, progressing successively from west to east. The first cut was taken out near the river, the draglines loading on to a track at the east. Meanwhile a second track was laid 150 ft. farther east, ready for use upon the completion of the first cut. Thus tracks were taken up and relaid in succession along the pit, 150 ft. apart, as the succeeding cuts were made. Figure 41 shows the layout of this job, the track system used in the borrow pit, and other things pertinent to this discussion. The two dragline machines traveled up one cut and down the next, one ahead of the other, being spaced about half the length of the borrow pit apart, so that one would reach the middle of the cut where

the other had started, as the other reached the end, and both would move to the new cut at about the same time.

To maintain maximum output one important thing was to keep the draglines supplied with cars for loading. To accomplish this it was the rule that an empty train returning for reloading would proceed first to the southerly dragline, and if a train were already loading there, wait its turn; there would then be a train

FIG. 42—BORROW PIT AND EQUIPMENT AT ENGLEWOOD DAM
This shows three of the dragline machines, some of the gravel trains, the derrick and coal pile, and some other items of equipment used at the borrow pit for the Englewood Dam.

loading at the other machine also. When the northerly dragline was completing the loading of next to the last car of its train, a whistle signal would be blown. At this signal the partially loaded train at the southerly dragline would move to the northerly machine, leaving its space for the next train. The time occupied in loading the last car was found to give time for the second train to come up, so that as soon as the loaded train at the northerly machine was ready to pull out, the train from

the southerly machine was ready to slip into its place. This system worked out so well that it was seldom during the course of the work that either of the dragline machines was without cars to load. In case the train at the southerly dragline finished loading first, it would proceed to the dam by means of a cross-over. The empty trains on their return trip passed the coaling and watering stations, and took on fuel as required. For full service, six 5-car trains were used.

Figure 43 shows one of the dragline machines in action. The distance from the old track location along the outer edge of the

FIG. 43—DRAGLINE MACHINE LOADING CARS

This is an electric machine with 85-ft. boom and 4½-yd. bucket. Under the conditions shown it was not unusual to load cars at the rate of a bucketful in 45 seconds. Three bucketsful loaded a car.

cut, to the present loading track, is about 150 ft. The extreme reach from loading track to foot of slope at the outer edge of the cut is about 170 ft. To make this reach the machine has to make nearly the full 180° swing. The depth of this cut is about 16 ft., about 4 ft. of which is below water.

Spotting the bucket over the car, so as to dump quickly without spilling the material, turned out to be much less difficult than had been anticipated, although, of course, it required a skilful operator. Digging under these conditions, with part of the material below water, and making the full 180° swing for

the outside material, it was not at all uncommon to load cars at the rate of a bucketful in 45 seconds. In fact, one operator loaded cars for a full 10-hr. shift at an average rate of 72 swings per hour.

The dragline machines worked two 10-hr. shifts per day. To illustrate the capacity of the machines under these conditions, and the amount of delay that may be expected on work of this sort, a reference to Table X., Chapter II., will show performance for one working season, April to December, inclusive. Out of a possible working time of 4,260 hours, one machine put in 3,935 hours of actual digging time, the lost time amounting to only 7.6%. The other machine with a possible working time of 4,230 hours was actually digging during 3,906 hours, showing 7.7% loss of time. During this period one machine loaded 70,550 cars—a total of 599,000 cu. yds., while the other loaded 69,141 cars—a total of 587,000 cu. yds., measured in the dam in each case. Seventy thousand cu. yds. per month for either machine was a fair month's work, and during one month one of the machines has a record of 84,000 cu. yds. The biggest month for the other machine was 77,000 cu. yds. The best record for one 10-hr. shift on this work was 2,329 cu. yds., or 274 cars. Table XI., Chapter II., gives an analysis of the causes of delays on each machine over a three-months period.

In taking an average over a long period it was found that the capacity of a 12-yd. car as loaded under these conditions amounted to about 8½ cu. yds. in the hydraulic fill embankment.

Borrow Pit Excavation at Germantown Dam. At Germantown the work was handled in a manner similar to that at Englewood, except that only one dragline machine was used in the borrow pit. This was a Lidgerwood electric machine with 100-ft. boom and 3½-yd. bucket. The borrow pit was long and narrow, being about half a mile long, but only from 200 to 450 ft. wide, and more irregular in shape than either the one at Englewood or at Huffman, hence the regular sequence of cuts could not be carried out as systematically as at the other places. The track system was so arranged that an empty train backed in as soon as the loaded train had pulled out. Only two trains were used, and with the two trains and one dragline machine it was not possible to keep the dragline as well supplied with cars as at Englewood. Nevertheless, comparatively little time was lost for lack of cars. Only two locomotives were available at Ger-

mantown, the difficult haul from the railroad, and the comparatively small amount of material to be moved (less than 800,000 cu. yds.) making it apparently inadvisable to go to the expense of shipping in a third locomotive for a spare. The water at all of these jobs was very hard and at Germantown it was sometimes dirty. This made it very difficult to keep the locomotives in continuous service during the working shifts, and boiler trouble was a menacing source of delay on this whole excavation job. By careful watching, emergency work between shifts, and more thorough overhauling on Sundays, however, the work was put through with comparatively little delay or interruption. A larger proportion of the excavation was below water level at Germantown than at Englewood, but this had little effect on the progress.

An average monthly run at Germantown was about 50,000 cu. yds. of material, and during one month approximately 70,000 cu. yds. were taken out.

Borrow Pit Excavation at Huffman Dam. At Huffman there was more gravel and less overburden in the borrow pit than at either Englewood or Germantown, and more of the excavation was below water level. The pit was located upstream from the dam as at the other places, and was fairly regular in shape, being about 1,600 ft. long and having an average width of about 1,000 ft.

The excavating machine was a Bucyrus electric dragline equipped with 100-ft. boom and 4½-yd. bucket, exactly like the electric machine at Englewood, except that the boom used was a little longer. Two trains served this machine, as at Germantown, but a third locomotive was available most of the time, so that the difficulties of keeping the two locomotives in continuous service were less severe than at Germantown. The successive cuts through the pit at Huffman varied in width from 125 to 150 ft. A series of tests was made with wider and narrower cuts to determine the most economical width. The wider cut has the advantage of less moving of tracks, while the narrower cut means less of a swing for the dragline, and therefore faster loading of the cars. It was the opinion on this job that for the dragline machine used here, a 125-ft. cut was the most economical width for this kind of work.

Table XII., Chapter II., gives the performance record of the dragline machine on this work during one working season,

January to November, inclusive. The biggest month's work was 70,600 cu. yds., while a good average month's work was about 60,000 cu. yds. The actual time lost during the full period was 21½% of the possible working time. Table XIII., Chapter II., gives an analysis of those delays. A comparison with the corresponding record at Englewood shows the advantage of the larger job where more flexibility in the transportation equipment is possible.

Following are representative cost figures for this borrow pit excavation at Englewood and at Huffman Dams, covering several million cu. yds. of material. The "Supplies" item includes coal, and electric power, as well as miscellaneous supplies. These figures include full depreciation for plant, and all costs properly chargeable to the work. Wages for dragline operators at this time were $250 to $275 per month, firemen 56c per hr., oilers 56c per hr., laborers 40c per hr., all men on an hourly basis being given time and a half after 8 hours, (11 hours' pay for 10 hours' work.)

Unit Costs of Borrow Pit Excavation

	Englewood Dam		Huffman Dam	
		Percent.		Percent.
Superintendence	$0.005	4.9	$0.005	4.7
Labor	.044	43.1	.039	36.8
Supplies	.033	32.4	.035	33.0
Plant Depreciation	.020	19.6	.027	25.5
	$0.102 per cu. yd.		$0.106 per cu. yd.	

Hydraulic Sluicing

Hydraulic sluicing was one of the major operations at two of the dams, and was carried on to some extent at two others. In all more than 2,000,000 cu. yds. of material were excavated by this method in connection with the hydraulic fill operations. At Taylorsville about 900,000 cu. yds were so moved, and at Lockington the quantity was approximately 800,000 cu. yds.; at Huffman and Germantown the quantities of hydraulic sluicing were about 210,000 and 100,000 cu. yds., respectively.

The material moved by this method included glacial till, hard blue clay, and sand and gravel. Usually, after being broken down in the borrow pits by the hydraulic monitors, commonly

known as "giants", it was sluiced to dredge pumps, to be pumped into the fill, but in one case, at Huffman, a considerable quantity was sluiced direct into the dam.

Hydraulic Sluicing at Taylorsville Dam. This method of excavation was decided on at Taylorsville chiefly because of the large amount of excavation for the outlet structure, suitable for use in the hydraulic fill, which lay at a sufficient elevation in the hillside so that apparently it could be moved to the dredge pumps by sluicing cheaper than by other methods. (See Fig.

FIG. 44—HYDRAULIC SLUICING AT TAYLORSVILLE DAM

The hydraulic giant is working at the foot of the bank, and the low pressure supply may be seen cascading down the face of the cut. The sluice ditch is in the foreground with the grizzly bars just in front of the pump house to reject sticks and rubbish and stones too large to be handled by the dredge pump.

44). After the excavation was opened up, the true character of the material turned out to be somewhat different from what had been assumed from the preliminary borings. Even so, sluicing was probably the best way to handle it even though the cost was higher than had been expected. As a matter of fact, when the limits of the excavation had been reached, and it was then a question whether to widen the cut into the hillside and thus borrow the additional amount necessary to finish the hydraulic fill, or to open up a dragline borrow pit in the river

valley, as was done at three of the other dams, it was decided to continue with the sluicing method. This decision was influenced somewhat by the fact that the hydraulic plant was already on the ground and that some of the equipment for the dragline-car method was not immediately available, but the concensus of opinion was that to continue sluicing would be the cheapest way to obtain the necessary material.

The surface material at Taylorsville was a glacial till,—a firm clay, with an admixture of gravel and rock fragments. On dry analysis about 60% of this material is held on a 100-mesh screen. Of the material passing the 100-mesh a large percentage is very fine, in fact the material as a whole yields a considerable surplus of fines for hydraulic fill dam work. This till at Taylorsville had a depth of from 15 to 25 ft. below the surface. Underneath the till was a very hard blue clay which upon being broken up gave about the same mechanical analysis as the till. The wash-boring samples gave no indication that it was very different except in color. This blue clay was very hard and tough, however, and could not be broken up by the giants except after being blasted. Even then it was hard to handle. Many of these clay lumps passed down the ground sluice, through the dredge pump, and on through a thousand feet or more of discharge pipe, and came out in the shape of firm, hard balls the size of one's fist.

When this blue clay was first encountered it was drilled by means of churn drills rigged up to be worked with a pulley and tripod. This was not entirely successful, and was rather expensive; later, "snake holes" were jetted into the face near the bottom, nearly horizontal, about 10 ft. deep and spaced about 10 ft. apart. A number of them were shot at the same time. This method was more effective and cheaper than the other, but it required a longer face in the pit, so that the drilling and shooting could be carried on at one place while the giants were working at another. Moreover, with a face more than 30 or 40 ft. high, this method tended to leave overhanging banks which were dangerous. At such places it was necessary to do some drilling from the top. During the latter part of the work well drills replaced the churn drills for this purpose. The well drill holes were spaced about 20 ft. apart both ways and were loaded with about ¾ of a pound of 40% dynamite per cubic yard. The mate-

rial was very hard to shoot properly, and none of the methods used were entirely satisfactory.

The water supply for these operations was taken from the river. For the giants, the supply was furnished at first by two 6-in. centrifugal pumps, each with a capacity of 1400 G.P.M. against 150-ft. head. These were connected up so that they could be run either in series, giving 1400 gallons at about 125 lbs.

FIG. 45—HYDRAULIC SLUICING AT TAYLORSVILLE DAM

This is a closer view of two of the giants working at the hard clay bank. The low pressure water may be seen coming down the face just beyond. The large giant at the right is operating under a pressure of about 175 pounds per square inch.

pressure, or in parallel so as to give about 2800 gallons at 60 to 65 lbs. pressure. For the low pressure water, to furnish the flow required in the ground sluice and a sufficient supply for the dredge pumps, a 12-in. centrifugal pump was installed, with a capacity of 5600 G.P.M., against 80-ft. head. This low pressure water was carried up to the top of the cut and allowed to flow over the face at various points, helping to wash down and soften the material in the face. After the blue clay was encountered it was necessary to increase the pressure for the giants, and two more 6-in. pumps were installed like those just described. These

two pairs of 6-in. pumps were then worked in parallel, the two pumps in each pair being connected in series. This gave 2800 gallons per minute, and supplied a 3-in. nozzle in the pit with a pressure of about 115 lbs. Later a small two-stage pump was installed with the suction taken off the low pressure water line, giving a chance to work another nozzle from this pump, and that worked so well, that eventually it was replaced by two 10-in. pumps, each with a capacity of 3500 gallons per minute at 180-ft. head, thus giving another line with a pressure of about 175 lbs. on a 3-in. nozzle, in addition to the high pressure line from the two pairs of 6-in. pumps.

The giants used at first were standard No. 2 hydraulic monitors with 2½ to 3-in. nozzles. Later a No. 4 giant with 2½ to 5-in. nozzles was installed for operation under the higher pressure. It eventually became the practice to work two of these giants close together (see Fig. 45), using the larger one for the heavier work, and the smaller for sweeping up the bottom and keeping the ground sluice clear. One of the secrets of success on this kind of work is to keep the nozzles close up to the face where they are much more effective than when working from some distance away. With a face as high as the one at Taylorsville, however, this is difficult because of the danger of caving of the banks.

The ground sluice at Taylorsville naturally took a grade of about 4%. For short periods when an unusually rich mix was being run, or when a large amount of sand and gravel was being carried, the sluicing would take a steeper grade temporarily, but would come back to about 4% under normal operation. The normal flow of water in this sluice was about 16 cu. ft. per second. The percentage of solids usually averaged about 8%, although at times they ran as high as 20%. This ground sluice was simply a well-worn gulley in the floor of the borrow pit, and its width was controlled at about 3½ ft. by means of plank sides staked down at places where there was a tendency for the flow to spread.

A portion of the excavation for the inlet channel lay at an elevation so low that it could not be sluiced to the sump. This excavation was taken out by a dragline excavator and most of it was thrown up into a pile above the elevation of the sump and where it could be reached by the giants. The same procedure was followed in handling some of the bottom excavation for the outlet channel.

This hydraulic sluicing at all of the dams was one part of the hydraulic fill job, and separate costs were not kept continuously on this portion of the work by itself. It has been possible, however, to get representative costs at most of the places where this method of excavation was used. The progress at Taylorsville varied considerably as the different classes of materials were encountered in the pit and as various methods were devised to meet the situation. The work was carried on by two shifts of 10 hours each. During the first season, before the installation of the additional high pressure pumps and the large giant, a good month's work was around 23,000 cu. yds., and the best month showed an output of only 28,500 cu. yds. After the installation of the large giant there was a marked increase in the monthly output, and during one month over 50,000 cu. yds. were taken out with the equipment here described. For short periods of a week or more, outputs of 150 to 200 yds. per hour were not uncommon, and quantities sometimes ran up to an average of between 200 and 300 cu. yds. per hour. On this kind of work, however, the average for a long period is very much less than the short time records would indicate. This is accounted for partly by the time lost in moving, and in trying to handle refractory material in the pit that was not thoroughly broken up by the shooting, but very largely because about the only way to tell the difference between a large and small output is by actual measurement. On most excavation jobs the rate of output shows for itself to an experienced observer, but with hydraulic sluicing the ground sluice looks about the same whether carrying a high or low percentage of solids. The difference between 6% and 12% solids in the ground sluice is hardly noticeable without close inspection, although the difference in quantity of material being moved is 100%. A job may run on for hours at a time, without it being realized that something has happened to cut down the output, unless given very careful attention by an experienced man. A skilful monitor man is absolutely essential for good results on work of this sort, especially in tough material like that described here, not only in order that maximum output may be maintained, but also that the pit may be kept in proper shape for continuous operation. To pick the soft places and make a big showing for a short period inevitably means a time of reckoning later on.

The following figures are given for two periods of work at Taylorsville: first, during the early work, before the best methods had been worked out for handling the hard clay, and before the additional high pressure equipment had been installed; and, second, for a period after the installation of the additional high pressure pumps and the large giant.

Cost of Sluicing at Taylorsville During the Early Period of Work

Quantity moved—243,500 cu. yds.

		Unit Cost	Percent. of Total
Superintendence and overhead.....	$ 9,475.03	$0.039	6.1
Labor	28,086.09	0.113	17.4
Electric power.....	10,324.68	0.042	6.5
Supplies	89,469.00	0.368	56.8
Plant depreciation.....	20,722.53	0.086	13.2
	$158,077.33	$0.648 per cu. yd.	

Cost of Similar Work at Taylorsville After Installation of Additional High Pressure Equipment

Quantity moved—164,800 cu. yds.

		Unit Cost	Percent. of Total
Superintendence and overhead.....	$ 3,620.77	$0.022	6.2
Labor	23,343.36	0.143	40.2
Electric power.....	9,802.31	0.060	16.8
Supplies	13,680.83	0.084	23.6
Plant depreciation.....	8,195.00	0.047	13.2
	$ 58,642.27	$0.356 per cu. yd.	

Labor was being paid 38c to 46c per hour, blacksmiths 60c per hour, monitor men 55c to 60c per hour, pump runners 50c to 55c per hour, all men on an hourly rate being given 11 hours' pay for 10 hours' work (time and a half after eight hours).

These costs include overhead, full plant depreciation and all costs properly chargeable to the work. The item for supplies includes the cost of explosives. These figures cover the cost of breaking down the material in the borrow pit and transporting it through the ground sluice as far as the dredge pump sumps. A comparison of these two sets of cost figures illustrates how costs may be reduced, in material of this sort, by the installation of sufficient high pressure equipment. The cost of power per

cu. yd. was increased 50%, and labor was increased slightly, but the net result was a marked reduction in unit cost because of increased output.

Hydraulic Sluicing at Lockington Dam. The borrow pit at Lockington was so located as to include a deposit of sand and gravel at one side of the pit while at the other side the material was mostly a blue clay. The sand and gravel, with its overburden of clay and loam, furnished a good percentage of the fines required for the hydraulic fill. Additional fines were needed, however, and the blue clay section of the borrow pit was drawn upon for that purpose. The sand and gravel was easily broken down in the pit by the hydraulic giants, and was good material to sluice except that it contained a rather high percentage of cobbles and boulders. The blue clay was more difficult to handle, although it was not as hard as that at Taylorsville, and did not require blasting. The material was so located that most of it could be sluiced to the sump on 4% grades, although at the latter end of the work these grades had to be flattened somewhat in order to secure all the material that was required, and some sand and gravel that lay too low in the pit to be sluiced to the sump, was thrown up to a higher elevation by a dragline machine.

The water supply for these operations was secured from the Miami and Erie canal, which passed not far from the borrow pit. The low pressure supply was taken to the edge of the pit by means of a ditch, and was distributed around the edges of the pit by lateral ditches which were built as needed. For the supply to the giants two 8-in. centrifugal pumps, each rated at 1950 G.P.M. against 100-ft. head, were set up so that they could be operated either in series or in parallel, or separately. In addition to these, a 14-in. centrifugal pump was set up on the line as a booster. With this pump on the line it was possible to operate two nozzles in the pit at pressures as high as 175 lbs.

As the work went on, it was found that more effective work could be done in the pit if the flow through the ground sluice could be increased. There was no more water available from the canal, but there was ground water seepage into the dredge pump sump, which not only forced the dredge pumps to handle more water than was economical, but raised the elevation in the sump, and therefore affected the grades at the lower end of the ground sluice. A 15-in. centrifugal pump, with a capacity of about 10 second-feet, was set up at a place where it would pull down the

elevation of ground water in the main sump, and the discharge from this pump was taken up to the ditch system carrying the low pressure supply. Thus about 10 second-feet of additional flow was continuously circulated through the ground sluice, adding materially to its transporting capacity.

The giants used at first were standard No. 2 hydraulic monitors with 2½ to 3-in. nozzles. Later a No. 4 giant with 2½ to 5-in. nozzles was brought in for use on the clay side of the pit.

FIG. 46—BORROW PIT AT LOCKINGTON DAM

This shows the sluice ditch in the Lockington borrow pit and the dragline machine piling up material which lay too low in the pit to be sluiced direct to the sump. By properly manipulating the ground sluice, it was possible to undercut this gravel pile and work the material down to the sump as shown in Fig. 47.

This large giant, operating at 175 lbs. pressure with a 2½-in. nozzle, broke down the blue clay very effectively.

As has been said, the ground sluices were laid out originally for 4% grades. It afterwards became necessary to flatten these grades in order to get more of the material from the bottom of the pit, and by careful handling these sluices were actually operated satisfactorily on grades as low as 2½% to 3%. The percentage of solids average from 4% to 7% over long periods, although at times very much higher percentages were carried

during short periods. There were many factors entering into this job, because it was a part of the hydraulic fill at the dam, which would not have been present had it been a straight sluicing job. The selection of suitable material, the proper proportioning of coarse and fine materials, the synchronizing of ground sluice discharge with the dredge pump capacities, all tended to slow up

FIG. 47—SLUICE DITCH UNDERMINING LOOSE GRAVEL

This illustrates the method of undercutting the loose gravel piled up by the dragline machine. (See Fig. 46.) By the use of the diagonal shear boards this ditch could be crowded into the loose gravel bank, thus undercutting the gravel pile and working the material down to the sump.

the sluicing operations in the pit, and to cut down the percentage of solids carried.

Towards the latter end of the job when it became apparent that the pit would not yield gravel enough for the completion of the dam, a dragline machine was installed, and passed up into the pit some material which lay too low to be reached by the sluicing. Figure 46 shows this machine and the pile of gravel which it has thrown up. At the time this picture was taken the ground sluice

was flowing along the edge of this pile of gravel. By properly manipulating the ground sluice it was possible to undercut the gravel pile and gradually work practically all of the material down to the sump without the use of giants at all. Figure 47 shows this process in operation and indicates how the sluice ditch could be crowded into the loose gravel bank by use of diagonal shear boards on one side.

The work in the pit at Lockington was carried on by two 10-hour shifts. A good month's work was between 50,000 and 60,000 cu. yds. The best month showed an output of 80,000 cu. yds. The average cost of this sluicing for the whole period of work was about 35c per cu. yd. To show what may be done in material like that at Lockington under favorable conditions the following costs are given for a period of four months, when the conditions were favorable for a large output and when no unusual delays occurred.

Cost of Sluicing at Lockington for Four Months Period

Quantity moved—280,600 cu. yds.

		Unit Cost	Percent. of Total
Superintendence and overhead	$ 3,500.00	$0.012	5.8
Labor	27,306.10	0.097	46.6
Power	16,353.72	0.058	27.9
Supplies	5,596.19	0.020	9.6
Plant depreciation	5,981.53	0.021	10.1
	$ 58,737.54	$0.208 per cu. yd.	

During this period the wages for laborers were from 35c to 40c per hour, blacksmiths 60c per hour, monitor men 55c per hour, pump runners 50c per hour, with eleven hours' pay for ten hours' shift, or time and a half after eight hours.

The above figures might be taken as representative for the approximate cost of a straight sluicing job in gravel, where there is a free getaway for the material and no probabilities of delays because of other dependent features of the job. They should not, however, be considered as representative of sluicing costs in connection with hydraulic fill dam construction, where the sluicing operations continually have to be arranged and controlled to suit the requirements of the hydraulic fill itself.

Hydraulic Sluicing at Huffman Dam. The main borrow pit at Huffman was so deficient in fines that the need of supplying this

deficiency became apparent very soon after the work started on the hydraulic fill embankment. A borrow pit was available on the hillside at one end of the dam, where the bedrock was overlaid with a deposit of glacial till and blue clay which apparently could be handled economically by hydraulic sluicing. A part of the material in this pit could be sluiced direct into the embankment of the dam. This is the only place on the Conservancy work where direct sluicing was feasible. From this pit about 80,000 cu. yds. were put into the dam by that method. The rest of this additional material, or about 130,000 cu. yds., was handled by sluicing to a dredge pump and pumping into the dam.

The water supply was taken from a channel leading from the river to the foot of the hillside slope. Two 10-in. centrifugal pumps, each with a capacity of 3500 G.P.M. against 180-ft. head furnished water to the giant. These pumps were connected in series and gave a pressure of about 125 lbs. on a 5-in. nozzle in the pit.

To consider first the portion of this material that was handled by direct sluicing. The material in the bank was broken down by the hydraulic giant and carried by a ground sluice to a convenient point in the pit, where it was collected and fed through a small V-shaped collecting basin into a line of 15-in. dredge pipe which took it out to the embankment of the dam. This pipe was used in preference to an open flume for several reasons: first, the pipe was available, and there were other uses for it on the job, so that the only charge against this work would be the depreciation; second, a pipe line is much more flexible than a flume, and is more easily handled as the embankment is raised; third, the pipe line could follow the natural grade of the hillside, eliminating the necessity of extensive trestle work, and could be carried out practically level on the dam embankment for long distances operating under all the head available, regardless of changes in slope.

The material in this hillside pit varied considerably as regards its clay content, and the material being delivered could be selected as desired. With a short length of discharge line, and therefore a steep average grade, the material containing the highest percentage of sand and gravel was selected in the pit. As the discharge line was lengthened material of higher clay content was sent down, until when the discharge line reached its extreme length the material sent down through it was about 80% clay.

This pipe line would carry such material satisfactorily on an average grade as low as 1¾%.

The small crew of men required on this work was a noticeable feature of the operation. The usual crew consisted of five men:

1 pump man looking after the two pumps,
1 monitor man, on the giant,
1 man on the open sluices in the pit, who also gave the required attention to the collecting basin at the pipe inlet, and
2 men on the dam at the discharge end of the pipe.

An average month's work in this pit was about 12,000 to 15,000 cu. yds. working two 10-hour shifts per day, as was done most of the time. The average cost of the 81,000 cu. yds. moved by direct sluicing was 49.8c per cu. yd., subdivided approximately as follows:

	Unit Cost	Percent. of Total
Superintendence and overhead	$0.018	3.7
Labor	0.142	28.5
Electric power	0.208	41.7
Supplies	0.019	3.8
Plant depreciation	0.111	22.3
	$0.498 per cu. yd.	

which includes not only the sluicing itself but also transporting through the pipe line and delivering on the dam. The cost of sluicing alone, taking the material only as far as the pipe inlet, would be about 44c per cu. yd. in this case.

The 129,000 cu. yds. of material in the lower part of the hillside pit was handled in the same manner as that just described as far as pit operations were concerned, but a dredge pump was required to boost the material on to the dam. This pump, instead of picking up the material out of the sump as was done in the main sluicing operations at Taylorsville and Lockington, was set into the discharge line from the pit, operating as a booster in that line. In other words, the material from the pit was fed direct into the suction of the pump. The distance from the collecting basin to the pump was about 400 ft., and the average grade of the line in this section was 4½%. The 100 ft. of line next to the pump was made up with airtight joints so that if necessary the pump could exert a suction pull through that part of the line to clear out an excess load of heavy material that had a tendency to settle in the line. Sufficient grade in the suction line was im-

portant. At first an installation was made with a grade of only 2¾%, which proved unsatisfactory due to the frequent clogging of the line.

Ordinarily, in an installation of this kind, the giant would not furnish water enough for most economical results in the ground sluice or at the dredge pump. This material, however, being mostly clay, was not particularly easy to break down with the giant; after being broken down it was easy material to sluice, and the ground sluices were short. For these reasons no addi-

FIG. 48—HYDRAULIC SLUICING AT HUFFMAN DAM
Showing a portion of the hillside borrow pit at Huffman where material was being sluiced direct into the dam.

tional sluicing water was required. At one time an additional supply was provided temporarily, but this was discontinued as apparently it was not necessary. The average grades of the ground sluice in the pit were about 4½%.

There was some trouble, of course, because of the direct feed into the suction of the dredge pump. The capacity of the collecting basin was small, and therefore gave very little regulation between ground sluice discharge and pump discharge. The dredge-pump man had to be on the alert every minute to avoid trouble from temporary variations in flow, due to spread or diver-

sion of the ground sluice in the pit, clogging, etc. As the discharge through the giant was uniform, and as that was the only source of water supply in the pit, it was possible for the dredge-pump man to operate quite satisfactorily in spite of the difficulties mentioned. The job was not large enough to warrant a real regulating basin at the inlet to the suction pipe.

FIG. 49—HYDRAULIC MONITOR NOZZLE AND DEFLECTOR
This is a close-up of one of the large monitors operating at a pressure of 125 pounds per square inch.

The working face in this pit was not more than 12 or 15 ft. in height. Therefore it was feasible to work close up with the giant, thereby getting the most effective work out of it. It was found that if the bank had receded 50 or 60 ft. back from the nozzle, the effectiveness of the giant was very much reduced. If this face had been much higher, so that the danger of caving had prevented the giant working so close, it is probable that the material in the pit would have required blasting. Figure 48

shows part of the operation in this hillside borrow pit, and Figure 49 is a close-up of the nozzle of the hydraulic giant operating at about 125 lbs. pressure.

The progress of work in the lower part of the pit averaged about 16,000 to 18,000 cu. yds. per month, two 10-hour shifts per day. This was somewhat better progress than the first work showed, largely because the men were broken in to the job, and some of the difficulties had been ironed out. The average cost of this 129,000 cu. yds. including sluicing to the dredge pump and pumping over to the dam, was about as follows:

	Unit Cost	Percent. of Total
Superintendence	$0.020	3.4
Labor	0.155	26.3
Electric power	0.261	44.3
Supplies	0.037	6.2
Plant depreciation	0.117	19.8
	$0.590 per cu. yd.	

The cost of sluicing alone, in this part of the pit, did not average more than 40c per cu. yd.

Hydraulic Sluicing at Germantown Dam. Sluicing operations at Germantown dam were an afterthought, and no consideration was given to them in the layout of the construction plant. After the work on hydraulic fill was well under way, it developed that the proportion of fines in the borrow pit material was hardly sufficient to furnish the amount required for the impervious core of the dam. Moreover, the dredge pumps had a capacity greater than that of the dragline machines in the borrow pit. Sluicing clay from the adjacent hillside into the sump would help both these conditions, and probably decrease cost as well as improve progress.

The amount of this clay that could be used, however, was limited, therefore an elaborate or expensive installation for getting out this material was not warranted. As a matter of fact, the installation was very simple. The giant at one end of the hog-box was supplied with water under 50 or 60-lb. pressure by means of an 8-in. pump with a capacity of 1950 G.P.M. at 140-ft. head. The pipe line supplying this giant was extended on up the hill to the pit, and at a convenient point along the line another 8-in. pump, of the same capacity as the one just mentioned, was

installed as a booster. This furnished 1950 G.P.M. at the pit with a pressure of about 80 lbs on a 2½-in. nozzle.

The material in the pit was a glacial till, very similar to the till already described at Taylorsville, and while neither the quantity of water nor the pressure available were as much as would be required for most economical sluicing, this layout answered the purpose very well, and furnished as much material as was necessary to meet the requirements. Because of the limited water supply available, and the low pressure, some blasting was necessary in the pit. The output varied considerably during different periods, largely because this pit was operated as an adjunct to the main pit, to fill in the deficiency of fine materials as required. The maximum monthly output was about 16,000 cu. yds. The unit cost varied from 20c to 50c per cu. yd.

One of the surprising outcomes of this hillside sluicing at Germantown was the demonstration that the power consumption at the pumps, when they were carrying this additional clay from the hillside, was no greater than when the hillside borrow pit was not operating. In other words, the lubricating effect of the clay in the discharge line from the dredge pump, at least made up for the power required to handle the additional load.

It should be kept in mind, while considering the methods and costs here described, that none of these jobs were operated under favorable conditions as sluicing jobs. In every case they were essentially a feature of hydraulic fill dam construction, where the selection of materials, or quantities handled, were dependent primarily on the requirements at the dam. None of the materials were ideal for sluicing operations; most of them were just the opposite. Moreover, certain selection of materials had to be made in the pits, to furnish more fines or less fines, according to the demands on the dam, which always meant higher costs in the pit to a greater or less extent. Again, whenever a shut-down was necessary at the dredge pump, or a suction or discharge line was plugged, the sluicing had to stop immediately until the pump could start up again. Hence, much loss of time, and added expense, resulted from causes outside of the sluicing job itself. These figures are given therefore, not as examples of typical sluicing jobs, but rather to show what can be done by sluicing, under conditions not by any means favorable, and still compete successfully with other excavating methods. It is doubtful if,

in any of these cases, the work could have been done as economically by any other method, and still fit in with the rest of the job.

RIVER CHANNEL EXCAVATION

To complete the river channel excavation work at the various towns and cities in the District required the removal of over 5,000,000 cu. yds. of material. Practically all of this excavation was handled by draglines similar to those used at the dams. While there was a considerable amount of this type of excavation at each of five towns, methods were quite similar at all of them, and therefore a detailed account is given only of the work at Hamilton. This is followed by mention of such methods and equipment at the other towns as were not of the standard type.

Channel Excavation at Hamilton. At Hamilton it was necessary to widen, deepen and straighten the old channel of the Miami River for a distance of about 2½ miles. The improved channel varies in bottom width from 800 feet to 500 feet, and the slope of the channel is 0.08 percent for the upper 6000 ft. and 0.05 percent for the remainder. There was a large surplus of excavation which had to be disposed of in two spoil bank areas, one at the extreme upper end of the job, the other near the lower end.

The choice of equipment for the 2,000,000 cu. yds. of excavation depended on the nature of the material, its position in the channel, the location of the spoil banks and the location of levees adjacent to the channel. Practically all of the material to be excavated was heavy gravel. As only two spoil bank areas were available, the average haul for excavated material was quite long—about 5000 ft. for material hauled to the south spoil bank, and about 4000 ft. for that sent to the north area. Figure 50 shows the general layout of work and the location of the spoil areas. After consideration of the hydraulic dredge, the dipper dredge and the steam shovel, and rejection of these types, it was decided that the dragline offered the best means of handling the job. Four draglines were used for excavating, with standard gage railway equipment for transporting the material to the spoil banks. Most of the channel excavation was done by a class 24 Bucyrus electric machine mounted on skids and rollers. This machine was equipped with a 100-ft. boom and 3½-yard or 4½-yard bucket.

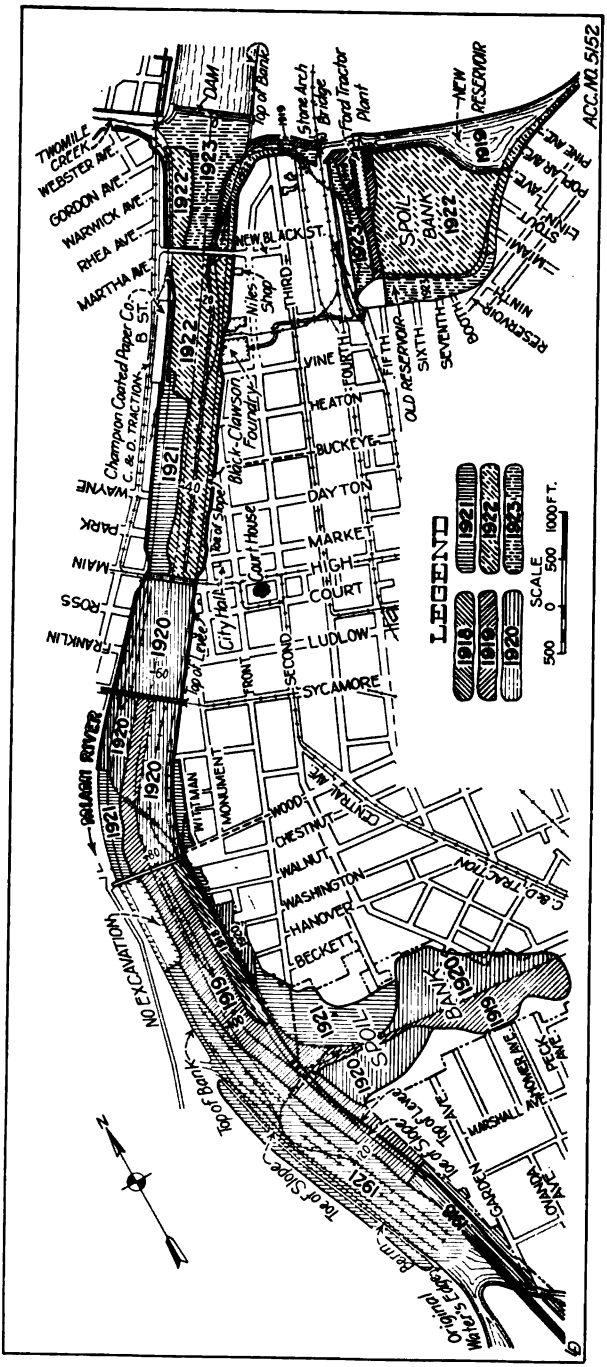


FIG. 50—CHANNEL EXCAVATION AT HAMILTON
Showing channel excavation, spoil banks, etc., also the order of work by years.

The first work done was deepening the river channel around and through a large island at the lower end of the channel improvement job. This was done to lower, as much as possible, the water level of the river above. The dragline started in about midway between the ends of this channel, worked downstream to the lower end, and completed the channel on the return trip. This channel was 45 ft. wide on the bottom, and the depth of cut was from 16 to 21 ft. through the main portion of the island. However, for a considerable distance the cut was very shallow. The total yardage excavated from this channel was about 90,000 cu. yds.; the best month's run was 68,000 cu. yds., this in heavy cut.

About 5 months were spent in doing this job, but the result justified the work, for the water level in the channel above was lowered at least 6 feet.

While the Class 24 dragline was digging along the east side of the river below the south spoil bank and casting directly into levee fill, construction of the tracks to the spoil bank was under way. A Class 14 Bucyrus steam dragline, caterpillar mounted, and equipped with a 60-ft. boom and 1½-yard bucket was used to construct the track embankment. By the time the Class 24 machine had proceeded upstream to a point where it was necessary to dispose of excess material, the transportation equipment was ready to haul into the spoil area. This area was about 3000 ft. long and averaged 1000 ft. in width, with its long axis practically at right angles to the river channel. Filling was started at the end farthest from the river on the left side of the main track, which extended about along the center line of the area. The average depth of fill in the spoil bank was about 10 ft. Throughout the job standard gage 50-ton saddle tank locomotives hauled trains of from 10 to 13 loaded 12-yard air dump cars to the spoil bank. The work was continued upstream along the east side of the main channel of the river by the Class 24 machine, assisted at times by the Class 14 machine, to the upper limit of haul to the south spoil bank. This was about 6000 ft. upstream from the spoil area.

The procedure from this time on was governed largely by the seasonal fluctuations of the river level. The work was arranged so that as nearly as possible the higher cuts away from the river were taken out during the high water season, the work nearer the middle of the channel being done in dry weather, usually the summer and fall months. (See Fig. 51.)

When the excavation on the west side of the main channel was started, it was necessary to build two trestles across the river to enable the trains to reach the spoil bank. These were pile trestles; the decks of each were fastened together by old cable and tied to one bank of the river as a high water precaution. The upper trestle crossed the river diagonally and was low, the track being about 6 ft. above low water. The other trestle was

FIG 51—CONSTRUCTION ACTIVITIES AT HAMILTON

This shows the dragline machine excavating the river channel during the low water period, loading material into cars to be hauled away to spoil banks. The section of river channel near the shore has been taken out during a period when floods might occur and the excavating equipment was safer working closer to the shore. In the immediate foreground is shown the movable concrete plant for placing revetment on the slopes. (See Fig. 102.) In the distance may be seen the new Black Street Bridge.

built opposite the main track leading to the south spoil bank and crossed the channel approximately at right angles. It was necessary to build this trestle much higher than the upper one in order to secure a proper grade for hauling. Little trouble was had with the low trestle due to high water, though it was overtopped several times. The other trestle, being higher, collected a great deal of drift, and several bents were lost due to high water even before the deck was built.

The embankment for the necessary tracks laid during the excavation on the west side were built by a Model 21 Marion steam dragline, caterpillar mounted, with a 32-ft. boom and ¾-yard bucket. This machine was used also to shift track on the spoil banks and for much miscellaneous work. In the larger cuts where practically all of the excavated material was hauled to spoil bank, track layouts were made so as to facilitate handling of trains with minimum delay to the dragline. Excavation on the west side was handled by working successive cuts up and downstream from the center of the channel westward toward the bank. Tracks were so laid that two adjacent tracks were always available, one for loading and the other for accommodating empty trains. Cross-overs were laid where necessary so that empty trains could pull over behind the ones being loaded; as soon as a loaded train pulled out, the empty one was on hand and there was no interruption to dragline operation. The upper trestle was removed by the dragline as it worked across the river south of the trestle. The lower trestle and approaches were removed after completion of all excavation which was to be hauled into the south spoil bank.

The end of the third construction season saw the completion of all channel excavation below the High Street bridge. A total of 1,000,000 cu. yds. of material was moved into levees or spoil bank during this period, practically all of which was excavated by the Class 24 Bucyrus machine. During five months this machine moved a total of 378,000 cu. yds., an average of 75,600 cu. yds. per month. The best month's run for this period was 82,000 cu. yds. The average depth of cut on this section of the work was from 5 to 15 ft.

Work on the upper section of the channel improvement was started by the electric dragline soon after completion of the lower section. Prior to this time about 56,000 cu. yds. of channel excavation had been done by contract. This material was excavated by a Marion steam shovel and was hauled to the spoil bank in 4-yard dump cars by 16-ton dinkey locomotives operated on a 3-ft. gage track. The spoil bank for this material (and all spoil above the High Street bridge) was located at the upper end of the job and about 2500 ft. east of the river. Some work had been done on the west side of the river by the Class 14 Bucyrus on caterpillars, the bulk of the material being cast into levee embankment, and the excess cast into the middle of the river to be

picked up later by the electric machine. The Class 14 machine then proceeded to build the necessary track embankment on the east side of the river, leading to the upper spoil bank, so that everything would be ready for the larger machine when the major channel excavation was started. Track embankment was carried about 5 ft. above low water and 30 ft. wide on top for double track. It was so located that the center line of the east track was about 180 ft. from the top of the east bank. There were about 40,000 cu. yds. in the main portion of this track embankment.

The Class 24 Bucyrus machine made practically all of the channel excavation on the upper section of the job in two seasons. There were heavy cuts along the east side running as much as 25 to 30 ft. deep in places. All of the material in this section was moved without requiring relocation of the tracks; the machine moved upstream between the tracks and the east bank, then returned on the river side of the tracks, taking out the cut in the middle of the river. The track embankment was removed later by the same machine. It moved along the bottom east of the embankment, loaded the material on 4 or 5-car trains and tore up the track, placing rails and ties where they could be hauled away by teams. In a 6-month period 378,210 cu. yds. were excavated by the electric dragline, the average being 63,000 cu. yds. per month. The best month's run was 90,660 cu. yds.

High water during the spring, after the lower section (that part below High Street) had been completed, deposited considerable gravel in the completed channel below High Street, due to the decreased velocity when the gravel laden water from the incompleted work above, reached the new channel. This is one of the disadvantages of working upstream on a channel excavation job. It was considered when the work was started, but the advantage gained by lowering the water level of the river by working from the lower end was thought to outweigh this difficulty. Another reason for working upstream on this job was the necessity for co-ordinating the work of excavation with that of the removal of a number of factory buildings on the east bank of the river north of High Street bridge. These jobs required from two to three years and the excavation could not be undertaken in their vicinity until they were practically completed. Figure 52 shows a cross section of the river channel opposite the plant of The Champion Coated Paper Company, where it was

necessary to remove the entire plant of The Sterling Paper Company in order to secure sufficient waterway. The gravel deposit referred to amounted to about 150,000 cu. yds. It was removed by the Class 24 Bucyrus machine, after completion of the heavy work above High Street. Two cuts were necessary, one following the deep portion of the channel and a second adjoining this on the east. Upon completion of this work the track embankment was dug out by the dragline, which moved along the bottom east of the embankment.

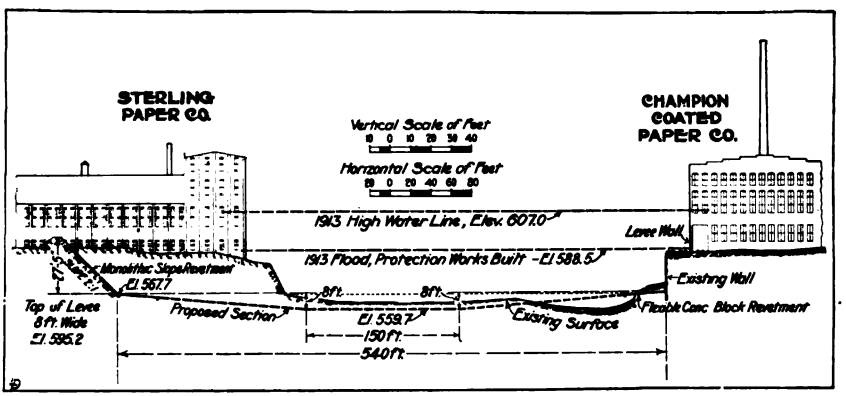


FIG. 52—CROSS SECTION OF RIVER CHANNEL AT HAMILTON

Showing a restricted condition of the river channel where it was necessary to move a whole manufacturing plant to make sufficient room for the handling of floods. The cross section of the improved river channel is indicated, the retaining wall on one side where there is no room for a levee, and the new levee with paved slopes on the other side. Also note the high water elevation during the flood of 1913, and the maximum height to which a repetition of the 1913 flood would reach with the flood control works in operation.

A cut-off channel joining Fourmile Creek to the river above the main channel improvement was constructed by a Class 14 Bucyrus steam dragline, mounted on skids and rollers, and equipped with a 60-ft. boom and 1½-yard bucket. The channel has a bottom width of 100 ft. and is 2000 ft. long. The total excavation was 92,000 cu. yds., and all the material was wasted at the sides of the channel.

Total channel excavation made by the Class 24 Bucyrus machine was 2,548,000 cu. yds.; of this amount 1,780,000 cu. yds. were classified as "pay excavation," the remainder being material

excavated beyond payment lines and work necessary for moving the machine and track. The above figures are for material moved once. Some of this material was moved a second time in order to get it into final position or loaded onto cars. Total "pay" channel excavation at Hamilton amounted to 2,142,000 cu. yds.; this figure does not include the 92,000 cu. yds. in the Fourmile Creek cut-off. Below are given the cost figures for the channel excavation by the Class 24 Bucyrus machine for the period June to October, 1921, when 378,000 cu. yds. were removed on the lower section of the job, and for the period January to July, 1922, when 378,210 cu. yds. were removed on the upper section. The figures include the cost of hauling, dumping and spreading. The dragline was operated on two 10-hour shifts per day. Wages varied somewhat, but during the periods referred to dragline operators were paid $250-$278 per month, oilers 50c-56c per hr., locomotive runners 50c-55c per hr., brakemen 40c per hr., and laborers 35c-40c per hr. The hourly rates are for an 8-hr. day. Workmen were paid for 11 hrs. when working 10-hr. shifts.

Tabulation of Costs

A.—Channel Excavation on Lower Section (378,000 cu. yds.).

		Unit Cost	Percent. of Total
Superintendence and Overhead	$ 5,119.38	$0.013	3.1
Labor	87,673.54	0.233	55.5
Supplies	38,445.87	0.102	24.3
Power	5,073.40	0.013	3.1
Plant Depreciation	21,847.60	0.059	14.0
Total	$158,159.79	$0.420	

B.—Channel Excavation on Upper Section (378,200 cu. yds.).

		Unit Cost	Percent. of Total
Superintendence and Overhead	$ 9,985.22	$0.026	5.0
Labor	103,531.66	0.274	53.0
Supplies	46,848.31	0.124	24.0
Power	7,191.08	0.019	3.7
Plant Depreciation	27,383.05	0.074	14.3
Total	$194,939.32	$0.517	

The unit cost of the excavation on the Fourmile Creek cut-off job was 19c per cu. yd. At the time this work was done dragline operators were paid $250 per month; oilers, 46c per hour.; firemen, 46c per hr., and laborers, 30c per hr.

FIG. 53—EFFECT OF CHANNEL IMPROVEMENT AT HAMILTON
The upper view shows the condition of the river before the work was started, and the lower one shows the completed work at the same location.

The final unit cost of excavating and disposing of all channel excavation at Hamilton, a total of 2,234,642 cu. yds., was $0.643 per cu. yd., showing the effect of winter work, lean work, and long time averages, as compared with the figures given in the preceding tabulations.

Channel Excavation at Dayton. The second largest channel excavation job, in terms of yardage moved, was that at Dayton. The length of improved channel there was 6.1 miles, and 2,146,000 cu. yds. of material were moved—this quantity represents "pay" excavation. The nature of material excavated was gravel similar to that at Hamilton. In general, the work was handled in much the same manner as at Hamilton, with this exception, that at Dayton the channel excavation was carried forward from the upstream end of the job, while at Hamilton work progressed upstream from below. The reasons for working downstream at Dayton were: first, it was desired to maintain flotation for scows in the upper section of the work, and, second, this method of procedure prevented the formation of gravel deposits in the completed channel. The depth of cut on the Dayton job was less than at Hamilton, averaging about 3.6 ft. Very little of the excavated material at Dayton was hauled to spoil banks by train—special methods were devised, as described later.

The excavating equipment consisted of four machines. Two Class 175 Bucyrus electric draglines, operating with 125 to 135-ft. booms and 3½ to 4½-yard buckets, and mounted on rollers and trucks, did all of the heaviest work. A Lidgerwood Class "K" electric dragline, mounted on rollers, and equipped with a 100-ft. boom and 2¼ to 4-yard bucket, was used for unloading scows and work around the spoil banks and for digging a cut-off channel at the lower end of the job. For miscellaneous small jobs and work in places where the larger machines could not conveniently operate, a Class 9½ Bucyrus steam dragline was used. This machine was caterpillar mounted and equipped with a 60-ft. boom and 1 to 1½-yard bucket.

Figure 54 shows the general layout of channel excavation at Dayton. The work above Washington Street was quite different from that below this point, and the job naturally fell into two main sections. Practically all of the excavation above Washington Street was shallow, and a large amount of the material had to be transported to spoil banks. There were three of these

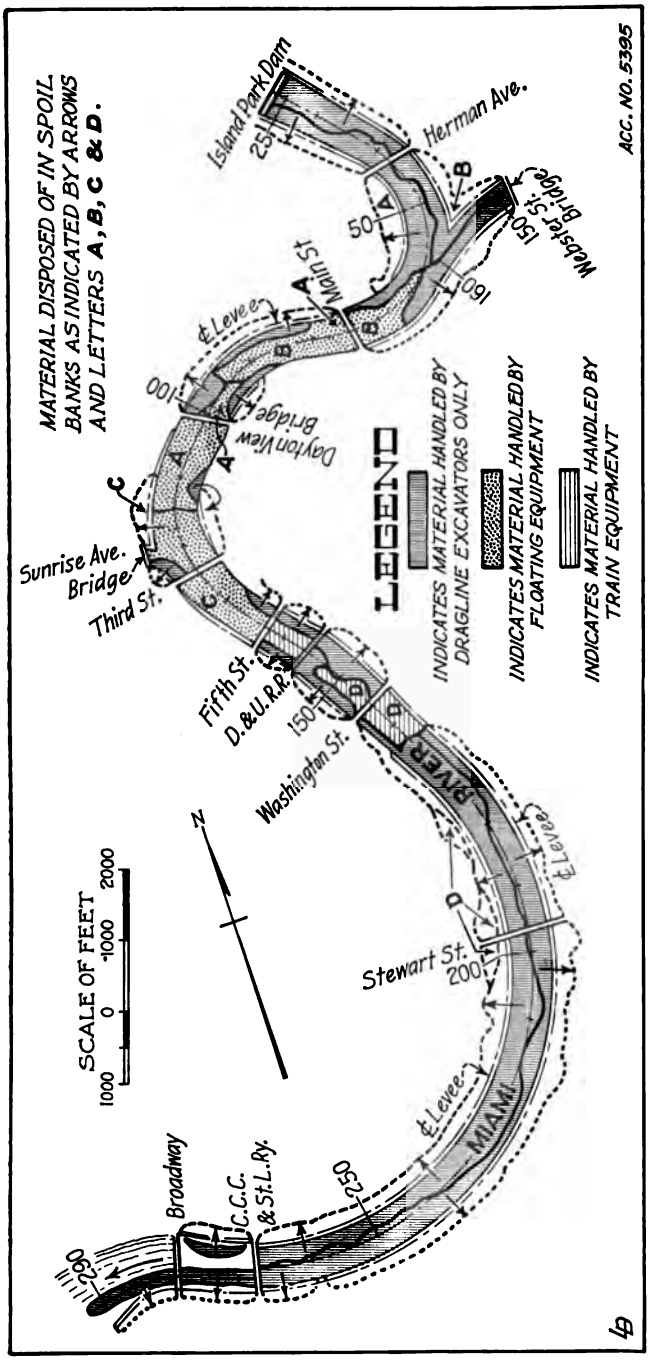


FIG. 54—CHANNEL EXCAVATION AT DAYTON
Showing channel excavation and methods of handling, location of spoil banks, etc.

areas, two near the mouth of Mad River and one at the mouth of Wolf Creek.

Due to the presence of high walls and levees along the river banks and numerous bridges, it was not thought feasible to haul the waste material by train through that section of the job between Fifth Street and the mouth of Mad River. Therefore floating equipment was devised and constructed for this work. (See Fig. 10, Chapter II.) The hauling equipment consisted of three 40 by 120-ft. scows and a 20 by 70-ft. steam tug. One of the Class 175 Bucyrus machines was mounted on a 40 by 80-ft. scow, and operated thus, with a 100-ft. boom, over a large part of the work. Material excavated by the draglines was loaded on the scows and they were towed to the spoil banks and there unloaded by another dragline. A temporary dam was built to assist in maintaining sufficient depth of waterway for the floating equipment. When operating the dragline dredge a path was first established as close to the side of the old channel as the scow could be floated. Then, starting at the downstream end of the run, the excavation progressed upstream, cutting to channel grade as far as could be reached toward the shore. Having reached the upper end of a run, the machine dropped back to the lower end, and floating in the newly excavated portion of the channel, worked upstream again.

On part of the work the dragline worked from the land, loading scows in the "scow channel." (See Fig. 11, Chapter II.) Where the river was so wide that all of the material could not be placed on scows with one handling, two runs of the dragline were made. At one point a mixture of clay and loam was encountered in the channel, and this material could not be satisfactorily handled on the scows, as it would slide off the decks. To handle this situation a hole was excavated in the "scow channel" large enough to hold all of the unsuitable material and, after disposition of the material from the hole to the spoil bank, the clay and loam excavation was dumped into the hole.

Practically all of the material excavated near the spoil bank at the mouth of Wolf Creek was good quality sand and gravel. This material was sent to the spoil bank and run through a washing and screening plant and made available for concrete aggregate.

About 350,000 cu. yds. were excavated by the scow-mounted dragline. The average quantity handled per month was 19,000

cu. yds., and the best month's run was 55,400 cu. yds. An average scow load was about 200 to 225 cu. yds., and 7 or 8 loads per 10-hr. shift was the maximum.

On a section of the work extending from Fifth Street to a point about 800 ft. below Washington Street, 85,000 cu. yds. of the channel excavation was handled by a Class 175 Bucyrus machine and loaded on trains. The hauling equipment was similar to that used at Hamilton. The average monthly output on this part of the work was 24,500 cu. yds., and 48,800 cu. yds. were handled during the best month.

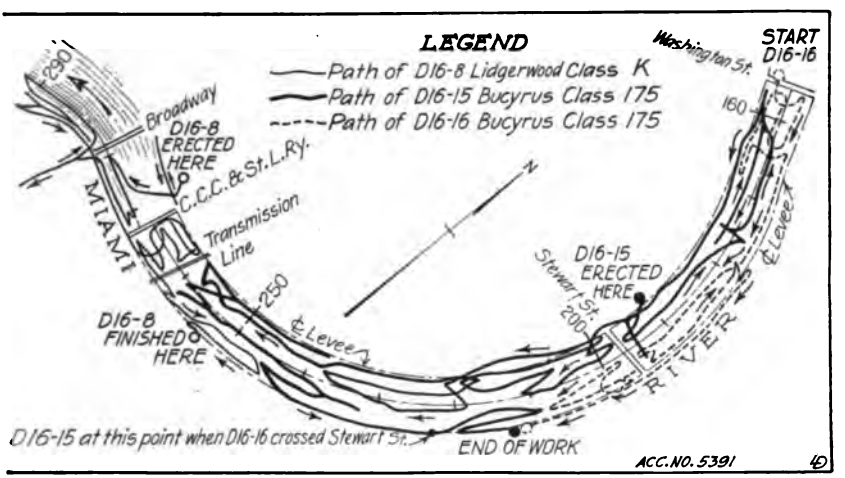


FIG. 55—DRAGLINE MOVEMENTS ON DAYTON CHANNEL EXCAVATION

This interesting sketch shows the paths followed by three dragline machines in handling more than 1,000,000 cubic yards of the river channel excavation at Dayton.

Channel excavation on the section of the work below Washington Street was handled by the two Class 175 Bucyrus machines, except for a small portion at the lower end. Cuts were heavier on this section than above. The channel was divided into two portions longitudinally, the material being so divided as to provide the most economical balance of work for excavation and embankment, excavation going into embankment directly opposite to its original position as nearly as possible. No hauling of material was done on this section. One-throw and two-throw

methods were used, and sometimes a third run was necessary. Figure 55 shows the paths traversed by the machines on this section. The total quantity excavated below Washington Street was 1,136,000 cu. yds.

At the lower end of the job a cut-off channel about 2500 ft. long and 75 ft. wide on the bottom, was constructed by the Lidgerwood machine. Total excavation for this channel amounted to 105,000 cu. yds.

The small amount of channel excavation in Wolf Creek was handled by the Class 9½ Bucyrus machine. Excess material was hauled to the spoil bank in 4-yd. side-dump narrow gage cars, hauled by steam or gasoline locomotives.

During a 16 months' period of operation, the costs of excavation *only* by the two Class 175 Bucyrus machines are shown below.

	Machine No. 15	Machine No. 16
Labor	$37,171.79	$37,314.58
Supplies	2,435.55	7,331.21
Minor repairs	6,613.39	4,137.07
Power	16,586.17	15,772.43
Cable	3,770.43	4,768.52
Miscellaneous	355.28	161.04
Depreciation	17,010.00	18,305.96
Total	$83,942.61	$87,790.81
Work done, cu. yds	981,989	641,510
Cost per cu. yd	$0.085	$0.137

The above costs contain all items chargeable to the work done and are based on the total gross yardages handled during the period, including second or third handling of material where necessary and estimated quantities of track fill. At the time the above work was done dragline operators were paid $250 to $280 per month; oilers, 50c to 56c per hr.; laborers, 40c to 46c per hr. resident electricians, $225 per month. Hourly rates are fo 8 hrs. work; payment for 11 hrs. was made for 10 hrs. work The cost of electric power was about 1.9c per k.w.-hr. on the District's lines.

Below are given cost figures for channel excavation at Dayto on the section of the work below Washington Street for a typica period of operation. During this period two Class 175 Bucyru

machines and the Lidgerwood Class "K" machine were operating. The quantity given is gross yardage — that is, for material handled more than once, each handling is included. Track fill is not included. The wage scale at the time this work was done was $250 per month for dragline runners; 46c per hr. for oilers; 46c per hr. for firemen; 30c per hr. for laborers; $200 per month for resident electricians. All men on the hourly basis were given time and a half after 8 hrs. The machines worked two 10-hr. shifts per day. All material moved was handled to the levees or spoil banks by the machines—no hauling was done. The work on this section was very well suited to dragline operation; cuts were fairly heavy and uniform and there were few obstructions, such as bridges, and no track laying.

Cost of Channel Excavation Below Washington Street, Dayton, for Typical Period

		Unit Cost	Percent. of Total
Superintendence and Overhead....	$ 7,613.96	$0.010	7.4
Labor	45,661.37	0.062	45.9
Supplies	14,320.37	0.019	14.1
Power	11,424.20	0.015	11.1
Plant Depreciation	21,205.71	0.029	21.5
Total	$100,225.61	$0.135	

The above, for a total of 740,000 cu. yds. moved.

The unit cost of excavating and disposing of all channel excavation at Dayton, a total of 2,146,000 cu. yds., was $0.637 per cu. yd. It cost about $0.76 per cu. yd. to dig and place material by the dragline dredge and scow method. (See Chapter II.)

Channel Excavation at Troy. Channel excavation at Troy amounted to 329,400 cu. yds., of which 108,400 cu. yds. were in a cut-off channel. All this work was handled by contract. The cut-off channel was about 4200 ft. long, 40 ft. wide on the bottom, and was cut with one side vertical and the other on a 2 to 1 slope. The work was handled by a Class 14 Bucyrus steam dragline, caterpillar mounted. The cut was made in two runs, part of the material being cast into spoil bank or levee, and part being hauled by train equipment. The contract prices for this work were:

Excavation,	material wasted at side	23c per cu. yd.
"	placed at side in levee	28c " " "
"	hauled to waste bank	48c " " "
"	hauled and placed in levee	53c " " "

The excavation in the river channel was handled in two sections, both by contract. The river channel was widened about 260 ft. through the central part of the city to provide an overflow channel. A low water channel 50 ft. wide was dug near the right bank of the river.

The section of improved channel which was dry excavation was handled by steam shovels and trucks or teams and wagons. With three No. 18 Osgood steam shovels operating and seven 7-ton trucks handling the hauling, 82,000 cu. yds. were moved at an average rate of 9,000 cu. yds. per month. The maximum for one month was 15,926 cu. yds.; this was carried in 8,237 loads of 1.93 cu. yds. each, average. During this month there was a total of 37 days' work by the shovels (3 shovels operating), and 124 days' work by the trucks (7 trucks operating). The contract price for this work was 65c per cu. yd., including hauling, dumping and spreading. Difficulty was had in dumping the trucks, but this was partly overcome by installing a false bottom which inclined towards the rear, thus making a nearly vertical slope for the material to slide on when dumping. The greatest difficulty was that of traction. The material excavated, and that over which the trucks travelled was a loamy silt or light sand. This condition, especially when the ground was wet, made hauling very difficult, and finally the trucks were abandoned and teams and wagons used for the hauling. With an outfit consisting of one No. 18 Osgood steam shovel and seven wagons hauled by three mules each, the remainder of this section of the job, amounting to 46,700 cu. yds., was handled. During a typical month, 8,200 cu. yds. were moved by this outfit; this material was hauled in 6,343 loads averaging 1.29 cu. yds. each. This was equivalent to about 43 loads per team per working day. On this job the average haul was about 500 ft. and the height of fill above the ground averaged 12 ft.

The wet excavation in the low water channel was handled by the Class 14 Bucyrus steam dragline used on the cut-off channel. The total quantity moved by this machine was 82,000 cu. yds.

Transportation

Borrow Pit Material at Dams. The transportation of material from borrow pits to dredge pumps was an important feature of the work at Englewood, Germantown and Huffman Dams, where a total of more than 5,000,000 cu. yds. of material was hauled.

The general system was about the same at all three places, with only such modifications as the topography or the nature of the work demanded. All the material was loaded into 12-yard, standard gage, air dump cars, which were hauled in 5-car trains by 40-ton, 4-wheel saddle tank locomotives. Track grades were limited to about 2% against the load, and curves were not sharper than about 300-ft. radius. Tracks were laid to line and grade, and were carefully kept up. Sixty to 70-lb. rail was used, with a tendency to favor the heavier rail because of the fact that it held its line better, and was in better shape for moving. Because of the use of dragline machines in the borrow pits, the spacing of the successive loading tracks was from 125 to 150 ft. apart, and one track location served a large quantity of material (approximately 10,000 cu. yds. per 100 ft. in most cases). Therefore more careful laying and maintenance of tracks was justified than might be the case in a steam shovel cut, or on a spoil bank where frequent moving of the track is a part of the regular routine. The tracks were laid out with a loop at the unloading end, except at Germantown where the topography would not permit. At the pit, a cross-over completed the circuit. That system, with a loop at one end and cross-over at the other, turned each train end for end each trip. This was an advantage, in that it resulted in dumping the cars on alternate sides each trip, whereby they were more easily kept clean than when dumped from one side all the time.

Track grades were important, and were laid out carefully. On a 2% grade a 40-ton locomotive could handle effectively a train of five loaded cars. The continuous service of a track maintenance gang was required to keep the tracks in good shape. At Huffman and Germantown this track gang consisted of a foreman and from five to ten laborers. At Englewood, where duplicate equipment was used, the track maintenance gangs were increased correspondingly. These track gangs also attended to the moving and relaying of the loading tracks.

Continuous repair work on cars was necessary. It was the practice to have a number of extra cars on hand, as there were always a number of cars out of service on the repair track. At Englewood where six 5-car trains were in service, the car repair gang consisted of two or three men, and at least eight or ten extra cars were kept on the job to permit the circulation over the repair track. At Germantown and Huffman where ten cars were

in continuous service, about four or five extras were required, and one man at each place was able to look after the repair work. It became standard practice to line the car bottoms with sheet metal as a matter of economy, and to facilitate clean dumping.

The difficulty of keeping the locomotives in continuous service has already been mentioned. At all of the jobs the water available was very hard, and in spite of the use of various boiler compounds frequent rolling and replacement of flues was unavoidable. Wherever it was economically feasible, an extra locomotive was held on the job to be put into service as needed, while one of the others was being overhauled. Where this was not feasible, most careful attention was necessary in order to maintain continuous service, and much and expensive emergency repair work was the result.

On all of the jobs the quantity of material put into the dam was limited by the capacity of the dragline excavators in the borrow pits. The transportation problem, then, was to so organize that the draglines would be kept supplied with cars and maintain a continuous feed to the dredge pumps. Two 5-car trains at Huffman and at Germantown served the one dragline at each place most economically. Train lengths were increased by one or two cars occasionally, but normally the extra cars were not needed. A third train might have saved some little delay at the dragline at times, but the extra cost was not warranted in the long run. At Englewood, where there were two draglines in the borrow pit with a third to be called upon if needed, six 5-car trains were kept in service most of the time. (The average length of haul was longer than at Huffman or Germantown.) Either train could serve either machine, and as a result of a carefully worked out system there was practically no waiting for cars at the draglines, and very little idle time for the trains.

Below are given detailed cost figures of the hauling and dumping of more than 700,000 cu. yds. of borrow pit material at the Huffman Dam, where the average length of haul was 2,000 ft.

	Unit Cost	Percent. of Total
Superintendence and overhead	$0.010	4.1
Labor	0.107	44.2
Coal	0.034	14.1
Other supplies	0.025	10.3
Plant depreciation	0.066	27.3
	$0.242 per cu. yd.	

These figures cover the whole period of work and include adjustments for winter work and miscellaneous expense. Labor was being paid from 40c to 46c per hr, locomotive runners 50c to 60c per hr, firemen 45c to 50c per hr, and brakemen 45c to 50c per hr. Coal was charged out to the job at an average price of from $5.00 to $5.50 per ton. Representative costs for similar hauling at Englewood were about as follows:

Average length of haul about 3000 ft.

	Unit Cost	Percent. of Total
Superintendence and overhead	$0.015	6.2
Labor	0.105	43.8
Coal	0.026	10.8
Other supplies	0.034	14.2
Plant depreciation	0.06	25.0
	$0.240 per cu. yd.	

Wages and coal costs were practically the same as at Huffman.

Coal consumption per locomotive was about two tons per 10-hour shift, under fairly favorable conditions. The cost of oil and waste averaged about $2.00 per train per shift.

Waste Material from Channel Excavation. The transportation of material from the river channel excavations to waste banks at Hamilton was similar in general to the work just described. The same type of equipment was used. The cars were loaded by dragline machines, as at the dams. Dumping the material into a spoil bank instead of into the hog-boxes, was the chief difference between this work and that just described.

At Hamilton the locomotives were heavier than those used at the dams, being 50-ton locomotives of the same general type as the 40-ton engines used at the other places. The average length of haul was about 5,000 ft. The limiting grades in the track layout were 1½%, and the sharpest curves were 300-ft. radius. At one place where a 1% grade was followed by a 350-ft radius curve, and then a 1½% grade, a 50-ton locomotive was able to handle a train of ten loaded cars. Part of the time on this work trains of 12 or 13 cars were used, but in such cases it was necessary to use a pusher on the 1½% grades. Sixty to 70-lb. rail was used in the track, and the track was kept in first-class condition, especially those sections of it which seldom required moving.

On the spoil banks a spreader car was used in order to avoid continual throwing of the track. This permitted moving the track over about 12 ft. at a time, and justified more careful maintenance of the track on the dump than otherwise would have been the case. A small dragline machine was used successfully for moving track on one of the spoil banks. This has been described more fully in Chapter II.

There was the same difficulty here in keeping locomotives in continuous service, as has been mentioned before, but an extra locomotive was available at Hamilton which could be used while another was being overhauled. One or two men were kept at work continuously making car repairs. Table XXX shows the cost of repair parts used on three 50-ton locomotives at Hamilton for a period of one year, during which time a total of 500,000 cu. yds. of material were hauled an average distance of about 5,000 ft.

Cost records are available on the hauling and spreading of 400,000 cu. yds. of material at Hamilton. This was for one season's work, but does not include the extra cost due to winter expenses, etc. On the other hand, it does include the cost of dumping and spreading on the spoil bank while the costs given for similar work at the dams have in them no charge for spreading.

Unit Costs of Hauling, Dumping and Spreading Waste Material From Channel Excavation at Hamilton

Average length of haul 5000 ft.
Total quantity handled 405,000 cu. yds.

	Unit Cost	Percent. of Total
Superintendence and overhead	$0.010	4.4
Labor	0.151	66.2
Coal	0.017	7.4
Other supplies	0.005	2.2
Plant depreciation	0.045	19.8
	$0.228 per cu. yd.	

Labor at Hamilton during this period was being paid 35c to 40c per hour. The coal used during this season cost only about $3.50 per ton on the job, while the average cost of coal used on the Hamilton job was approximately $5.50 per ton. In other words, the low price of coal used during this particular period cuts about 1c per cu. yd. off the cost of hauling. To bring this up to an

average for the whole job another 3c to 4c should be added to cover winter costs, unproductive work, idle time, etc.

Table XXX.

Supplies and Repair Parts for Three 50-Ton Locomotives for One Calendar Year

Oils and greases	$ 322.53
Lanterns, wicks, etc	31.40
Valves, cocks, etc	160.88
Gaskets, packing, etc	55.42
Injectors and parts	78.48
Brake heads	42.08
Brake shoes	114.80
Brake rod	10.00
Soft plugs	8.67
Driving box brasses	75.00
Brass bushings	9.20
Dick belt	60.06
Babbitt	3.84
Dump air hose	9.17
Fire hose	62.50
Steam hose	29.00
Boiler compound	86.63
Muriatic acid	15.00
Lumber	23.68
Paint	3.65
Steel, 53 lbs. 2-in	13.25
Flues and ferrules	344.98
Prest-o-Lite and oxygen	54.77
Sand dryer	65.25
Window glass	8.91
Waste	23.00
Tools (small)	63.89
Retreading 6 pair wheels	120.00
Miscellaneous, iron, bolts, etc	133.96
Total for 3 locomotives	$2,030.00

We then have an average cost of hauling, dumping and spreading at Hamilton of about 28c, for a haul of 5,000 ft., with 1½% maximum grades, 50-ton locomotives and 10-car trains, as compared with 24c at the dams, where the hauls were 2,000 to 3,000 ft., the maximum grades 2%, and where 40-ton locomotives were used with 5-car trains.

At Dayton about 500,000 cu. yds. of waste material from channel excavation was moved to spoil banks by means of scows and tug boat. This has been referred to in Chapter II, under the

heading "Dragline Dredge," and is illustrated in Fig. 10 and Fig. 11. The average length of haul was about 3,000 ft. An ordinary load for a scow was from 200 to 250 cu. yds. The unit cost of hauling by this method during a favorable period, not counting extra expense for winter work, delays, etc., was about 26c per cu. yd.

CHAPTER V.

EMBANKMENT CONSTRUCTION—METHODS AND COSTS

Definitions

During recent years, since the construction of hydraulic fill dams, and similar works by modified hydraulic fill methods, has become more common over the country, there has been a tendency to speak of any fill in which water is used for sorting or compacting the material, as an "hydraulic fill." Many dams which have been built by semi-hydraulic or puddled fill methods, have been designated loosely as hydraulic fill dams. Recently there has been considerable discussion of this matter in the technical press, with the purpose of deciding on suitable definitions to make proper distinction between hydraulic fill, semi-hydraulic fill, and other well known methods of building embankments. The following definitions are offered as meeting in a general way the ideas which are common in the minds of most engineers who have had to do with the design and construction of hydraulic fill dams:

> **Hydraulic Fill Dam.** One in which the materials are brought to the dam and laid down in the embankment by means of flowing water, and manipulated in such manner as to segregate the fines within a predetermined section to form an impervious core.
>
> **Semi-hydraulic Fill Dam.** One in which the materials are put into the embankment by some means other than by flowing water, and then worked over or manipulated by means of water in such manner as to separate out and segregate enough of the fines within a predetermined section to form an impervious core.

The dams of the Miami Conservancy District are true hydraulic fill dams, although relatively small portions of them were built by other methods, as will be seen by a study of this chapter. More than 7,000,000 cubic yards of true hydraulic fill was placed in these structures. Although the specifications provided for semi-hydraulic fill as one of the alternate methods of embankment construction, no true semi-hydraulic fill was placed on any of the Conservancy work, although as a matter of convenience

considerable quantities of fill that were placed by puddling and other similar methods, were classified in the Conservancy records under the item which included semi-hydraulic fill.

Hydraulic Fill At Dams

Stream Control. The control of the rivers at the dams during construction played an important part in hydraulic fill methods and progress. Until the flood control works were far enough along to give protection, the valley was in constant danger of another disastrous flood. The construction program had to be so arranged that by no possibility would this flood danger be intensified by the conditions existing at any of the jobs during a flood period. Both on this account and as a precaution against damage to uncompleted work, the river closures at the dams had to be so handled that when the closures were once started, they could be carried to a safe height before the next flood season; or if this were not possible, temporary spillways must be provided. The normal flood season in the valley was from January to March, inclusive. While a large flood might occur at other times during the year, it was extremely unlikely that one occurring at other times would be large enough to be disastrous. In planning the construction program, therefore, it was assumed that a flood as great as that of 1913 might occur during January, February or March, but that at any other time, a maximum flood would not be more than half that size.

At Lockington, Huffman and Taylorsville dams, where combination outlet and spillway structures were used, the handling of floods during construction was somewhat simplified. It will be recalled that the structures at these dams consist of two retaining walls facing each other, with a floor between, forming a concrete lined gap through the dam embankment. A weir across this gap forms the spillway, and the conduit openings are built through the base of the weir, at river level. (See Fig. 27, Chapter III, and Fig. 91, Chapter VI.) By postponing the construction of the cross weir until after the embankment was nearly finished, a gap of large capacity was made available for handling floods during the construction period. At Lockington the outlet structure was located at some distance east of the river channel. Still farther east was a sag in the natural ground which served as a natural temporary spillway. This was left open until the section of embankment to the west of the struc-

ture, including the river closure section, had been filled up to a sufficient height prior to the beginning of a flood season. Then, as soon as probability of a dangerous flood had passed for that year, the embankment east of the structure was begun, and both sections were easily brought up to a safe height before the following flood period arrived.

At Taylorsville the situation was complicated by the fact that in the old river bed, where the dam was to cross it, was a deep hole filled with mud and silt. This had to be cofferdammed off and cleaned out before the construction of that section of hydraulic fill could proceed. This increased very considerably the amount of work to be done in the river closure section between flood seasons, and made it essential that the rest of the dam be brought up first, to a safe height, leaving as short a section as possible to be put in across the river channel when the time for river closure came. Furthermore, since the excavation for the outlet structure at Taylorsville was suitable for use in the hydraulic fill embankment, the natural thing to do was to build a section of the fill to one side of the river channel while this excavation was in progress. As the location of the outlet structure was close to one end of the dam, a long section of fill across the valley floor on the other side of the river was built up first, leaving the river gap as narrow as was considered safe, and by this plan it was possible to clean up the bottom of the old river channel, and bring up that section of hydraulic fill, in one season, high enough to turn any possible flood through the outlet structure gap.

At Huffman the outlet structure was located so close to the river's edge that a temporary diversion had to be made on that account, so the gap to be closed there included the temporary diversion channel as well as a portion of the original river channel. There, as at Taylorsville, the outlet structure was located at one end of the dam. The volume of embankment was too great to permit its being brought up to safe height all at once, so the section beyond the temporary diversion channel was built up first, and after that had been carried to a safe height, the remaining section, making the river closure, was built up during one season.

At Germantown and Englewood, where the twin conduits were used, and the embankments are continuous across the valley, stream control was a somewhat more difficult problem. Addi-

tional waterway was provided by building the conduits deeper than was required for final capacities, giving them temporarily, for stream control purposes, more than twice their ultimate size. Even so, a flood like that of 1913 would back up so high behind these openings, that a large amount of fill had to be put in to prevent danger of overtopping. At Germantown the valley was narrow, and it was possible to build the required amount of fill during one season. At Englewood, however, the valley was so wide that even though a season's work was as much as 1,000,000 cu. yds., it was not possible to bring up the entire embankment to a safe height between flood periods. Even with a section of

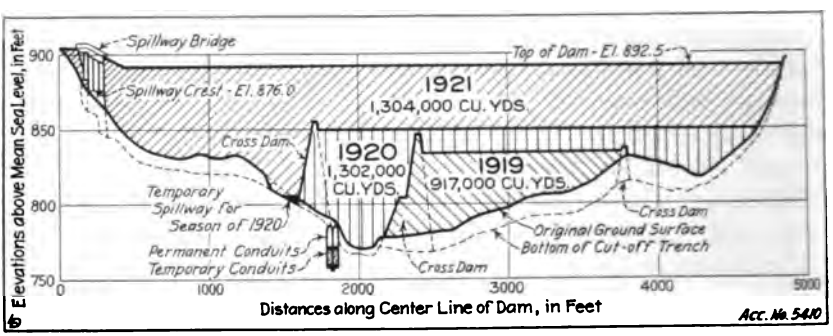


FIG. 56—STREAM CONTROL ENGLEWOOD DAM

This is a longitudinal section of the dam showing the location of the outlet conduits with respect to the river channel, the location of the temporary spillway, and the program of work by years so as to provide safe control of the river during construction.

embankment brought up to a sufficient height on one side of the river, it still was not possible to close the river section and bring up the rest of the fill in one season. The yardage was too great. Therefore, while the closure of the river channel itself was being made, a temporary spillway was provided to take care of any floods that might occur during that period of the work. The floor of this temporary spillway was about 35 ft. higher than the original river bed, so that the closure of that section, when the time came, was that much easier. Figure 56 is a longitudinal section of the Englewood Dam, showing the location of the outlet conduits with respect to the river channel, the location of the temporary spillway, and the sections of embankment placed

during three succeeding seasons in order that the river control might be handled safely.

This dividing of the hydraulic fill embankment into sections required the construction of dams across the ends of those sections, to retain the hydraulic core. These cross dams needed to be very carefully constructed as each became a permanent part of the completed structure. Obviously they could not be built by hydraulic fill methods, but the selection and compacting of material in them must be so handled that they would become an integral part of the whole structure, without introducing elements of weakness due to settlement or to lack of water tightness. These cross dams were built either by puddled fill or rolled fill methods and will be referred to again under those headings.

Hog-boxes. The methods of obtaining the material from the borrow pits and bringing it to the dams have already been described. At Germantown, Englewood and Huffman, where hauling equipment was used, an elongated bin or trough was provided into which the material could be dumped from the cars, and from which it was sluiced to the dredge pump. This structure, called the "hog-box," had a length of about 150 ft., was about 18 to 20 ft. wide at the center and about 15 ft. at the ends. The dumping track came in along one side, at an elevation about 10 to 12 ft. above the bottom of the hog-box. There was a small opening controlled by a sliding gate, in the center of the side opposite the dumping track, and through this opening the material was sluiced to revolving screens to take out the oversize, and thence to the pump sump. The floor of the hog-box sloped both longitudinally and transversely, towards this opening, the slope from the ends being about 4%, and the transverse slope about 10%. Figure 57 shows the principal dimensions of one of these hog-boxes, and its relation to the other features of the hydraulic plant, and Fig. 26, Chapter III, is a photograph of one of them in operation.

It was here that storage was provided for the material coming from the borrow pits, and an opportunity was given of feeding this material at a uniform rate to the dredge pump. The material was washed from the hog-box to the pump by means of small giants working at about 50 to 60-lb. pressure. This gave the opportunity of thoroughly breaking up the material and mixing it with the water before it reached the pump sump. There was also the chance here to work out to one side large boulders or

lumps of clay which might cause trouble farther along the line. The storage provided at the hog-box was usually about an hour's run for the borrow pit equipment or the dredge pumps, so that if an interruption occurred at either place the other end of the work could go on.

The structure illustrated in Fig 57 is representative of the hog-box installation at Germantown and Huffman. At Englewood, however, where duplicate equipment was used throughout, the hog-box structure was larger; it was divided in the center by a partition, and had two openings in the corners, one on either side of the center partition, where the material was fed to the two dredge pumps. (See Fig. 26, Chapter III.)

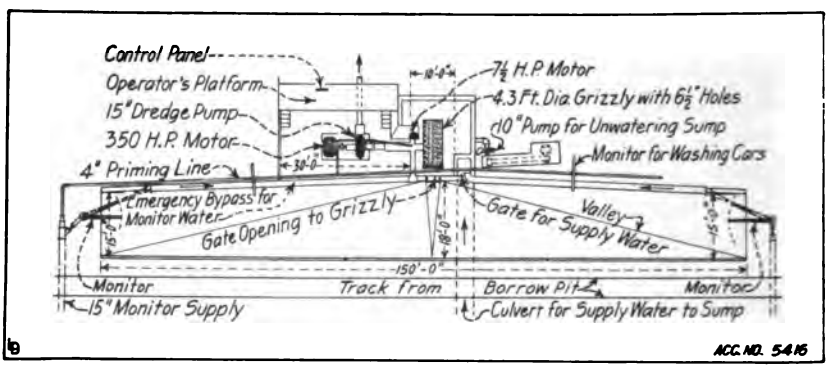


FIG. 57—SKETCH OF HOG-BOX SHOWING PRINCIPAL DIMENSIONS AND FEATURES OF DESIGN FOR A HOG-BOX SUCH AS WAS USED AT HUFFMAN AND GERMANTOWN DAMS

The one at Englewood was similar to these except that it was larger and designed to handle two dredge pumps instead of one. (See Fig. 26.)

The floors and sides of the hog-boxes were of concrete. The floor thickness was usually about 12 in., the bottom 8-in. of which was a very lean mixture. The wearing surface, about 4-in. thick, was made of rich concrete and was carefully troweled off to grade. It was also necessary to put a rich wearing surface on the exposed face of the wall forming the side of the box opposite the dumping track, as impact from flying pebbles caused constant wear. The wearing surface on the floor would take care of about 400,000 to 600,000 cu. yds. of material, after which it would need replacing. In one section of the hog-box floor at Huffman, while it was being repaired, hard 12 to 15-in. boulders

having one flat face were selected and laid in mortar with the flat side up. The mortar between the stones wore down about 1-in. in a short time, but from that time on there was no noticeable wear. This section was fully as efficient as the rest of the floor and undoubtedly much more durable.

For sluicing the material from the hog-boxes, small giants were located at each end; usually not more than one of them was used at the same time, except at Englewood, where duplicate equipment was used. The material coming in on the cars was fairly well broken up, but the action of the giant was beneficial in helping to complete the process. Usually a 2½-in. nozzle with 50 to 60-lb. pressure was found most economical. Occasionally when a hard chunk of clay was encountered it was an advantage to increase the pressure. It was necessary that the feed to the dredge pumps be fairly uniform in order to provide for a reasonably uniform operation of the pumps themselves, and therefore the process of sluicing from the hog-box was one requiring some skill and could not be handled in haphazard fashion. It was found best, usually, to play the stream along the toe of the material, so as to keep a regular amount moving toward the opening, and to prevent the sliding or sloughing of a large mass at one time.

Oversize Rock Screen. At all of the dredge pump sumps it was necessary to screen out the oversize rock or large chunks of clay which were too large to pass through the pump and discharge line, or which, by nesting together, might cause trouble in the suction. While much larger stones could be handled by the pumps if they came along one at a time, it was found best to reject from the sump all stones which would not go through a 7-in. opening. One thing that contributed to this decision was the fact that at all the dams a considerable amount of riprapping and paving was specified. The supply of suitable material was limited, and all the oversize rocks rejected at the sump were suitable for this purpose.

The first installations for the screening out of this oversize, were sloping grizzly bars over which the discharge from the hog-boxes or from the ground sluices was passed. The rocks and lumps held on these grizzlies were thrown out by hand. This method was crude, and required the continuous services of two or three men to keep the bars clear. A movable self-cleaning grizzly was worked out at one of the dams, and worked very well

except that the wear on sprockets and chains was very severe because of the grit in the water, from which it was almost impossible to protect them. The arrangement finally adopted at most places, was a revolving cylindrical screen about 4½ ft. in diameter and 9 or 10 ft. long, made of ⅜-in. boiler plate and perforated with 6½-in. circular openings spaced so as to give about three inches of metal between the holes. This screen was suspended over the sump, with its receiving end at the discharge opening from the hog-box. It was connected through a jack shaft and belts to a 7½-H.P. motor, and was geared down so as to revolve at a speed of seven revolutions per minute. The barrel of the screen was set on an incline of about ½-in. per foot. Everything dropped through the screen into the sump except the oversize material, which was discharged through the open end into buckets or cars, by means of which it was taken to storage piles, or direct to the riprapping or paving jobs. One man at the discharge end of the screen looked after the loading and taking away of the buckets or cars, and at the same time watched for clay lumps coming through, which he cut up with a shovel and threw back into the sump. In this way it was usually possible to keep the rock free from foreign matter and suitable for use as paving, without re-sorting. During periods when the clay came too fast to be thrown out in this way, a second man would be put on to help, or the rejections from the screen could be wasted until the material became reasonably clean again.

It was found that one of these revolving screens would pass approximately 500,000 cu. yds. of material, after which time the holes at the upper end would be worn to an extent that repairs were necessary. These repairs were made by putting another lap of boiler plate around the worn part of the screen and cutting new holes of the proper size. The holes were cut with an acetylene torch very quickly and inexpensively.

Incidentally, these revolving screens had some effect of equalizing the flow of material into the sump, that is, in preventing large masses from going in all at once. They also had the beneficial effect of further separating the material into its component parts, making it that much easier to handle in the sump and through the dredge pumps.

Sump. At the first dredge pump installations, the sump was simply a concrete-lined pit with a hopper-shaped bottom. The pump foundation was several feet above the normal water surface

of the sump, and the suction line extended nearly to the bottom. While such installations had been common practice up to that time, there were a number of objections, the overcoming of which would result in much more economical work. One objection was that the solid material in the sump had a tendency to pile up to the point of instability, and then suddenly slough down, burying the end of the suction pipe and often resulting in plugging it. When this happened pumping operations had to be shut down until the suction line could be dug out and the plug removed. The sump being below ground level, with no chance for drainage, this was not a simple operation.

Again, the suction was not efficient. The loss of head in the suction pipe was very high due to the character of material being handled, the necessary length of pipe, and the unavoidable bend in the line to the pump. In many cases the actual suction lift was only about one-third the total loss of head between sump and pump. Consequently some study was given to sump conditions, and at later installations where the conditions made it feasible, a revised sump design was used which eliminated most of the objections mentioned. First, the sump was elevated enough so that it could be drained and cleaned out from the bottom; second, the pumps were set in an adjacent chamber, with a waterproof wall between, and low enough so that the suction entrance to the pump was on a level with the bottom of the sump. A short length of pipe through the wall was all that was required for a suction line. There were no bends in it, and the receiving end was in the shape of a bellmouth to further improve its efficiency. The low pressure water (in addition to that furnished by the giants at the hog-box) was brought into the bottom of the sump directly opposite and on line with the suction line, and the revolving screen overhead was set in such position that all material dropped into the sump directly over the gap across which this low pressure water supply was being fed to the pump. Figure 58 illustrates the improved sump design and also shows comparison with the original layout. The new design eliminated suction troubles almost entirely, it cut down to a minimum the loss of head in the suction line, and it eliminated priming troubles entirely, as the dredge pump was under pressure whenever there was water in the sump.

Dredge Pumps. Fifteen-inch pumps were selected as standard for all the hydraulic fill operations. The determination upon

this size was influenced by two things: first, that a 15-in. pump which was fed to capacity would build up the embankment about as fast as it seems safe that a hydraulic fill should be brought up, giving reasonable time for compacting, and second, that a larger pump would have materially increased the difficulties of water supply and disposal. All these pumps were electrically operated, the motors varying in size from 250-H.P. to 500-H.P., depending on local conditions.

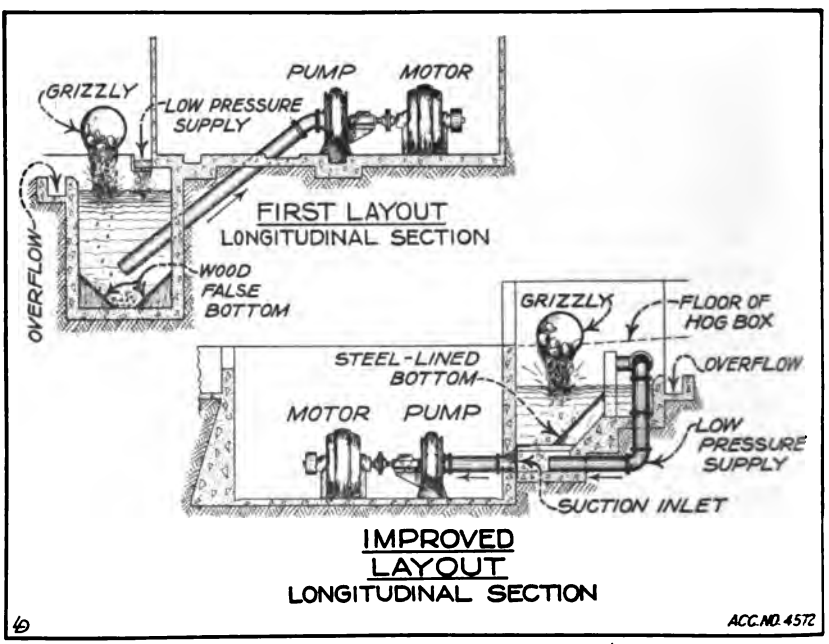


FIG. 58—IMPROVED SUMP FOR DREDGE PUMPS

The upper view shows the original installation, and the lower one the improved layout which gave much better results.

The first pumps installed had cast iron shells and runners. It was found that, with this heavy material which was being handled at the dams, these cast iron runners were good for only about 10,000 to 12,000 cu. yds. The replacement of runners became at once a source of serious expense and delay. The shells themselves were good for about 100,000 cu. yds., when they had to be either discarded or repaired. The first step towards im-

provement of pumping equipment was the substitution of manganese steel runners, and at the same time a design was perfected for removable manganese shoes for the runners, so that instead of discarding the whole runner when the tips of the vanes had been worn off, it was only necessary to put on a new set of shoes. In this way the life of the runners was increased to between 100,000 and 200,000 cu. yds.

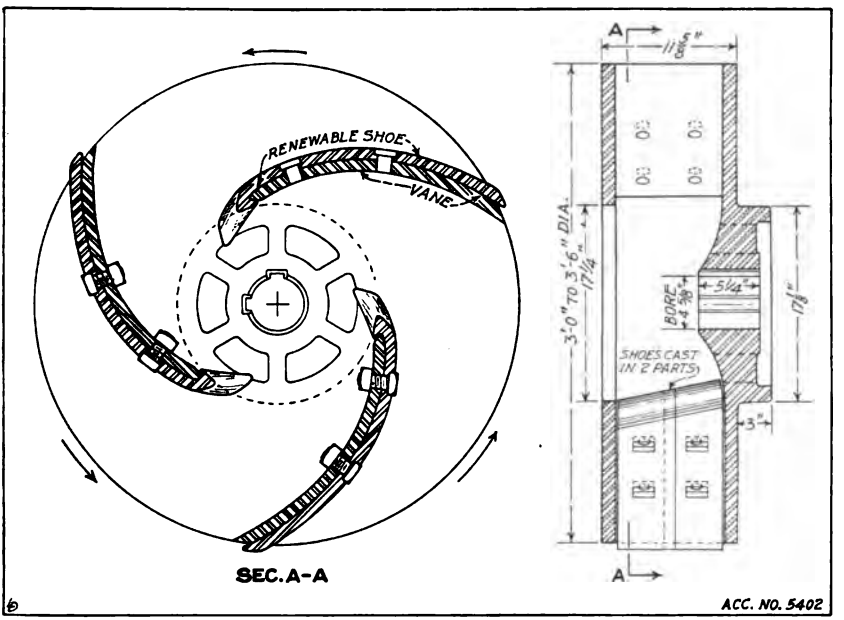


FIG. 59—STANDARD IMPELLER FOR DREDGE PUMP

These impellers were of manganese steel with removable manganese steel shoes on the vanes. The effective size of the impeller could be increased by replacing shoes with longer ones. However, it was found best not to allow a shoe to project more than 2 inches before changing to the larger size impeller.

The next step was the introduction of manganese steel shells, which gave more than double the length of service that the cast iron shells were good for, and later a full manganese steel pump of simple design was adopted as standard at all the dams. This was a simple pump without liner plates. Figure 59 shows the general design of the runner, which was worked out by the District's engineers in co-operation with the manufacturers, with the

idea of getting a runner that would be most satisfactory from the standpoint of efficiency, durability, and freedom from clogging. It was the practice to substitute these new pumps for the original ones at each of the dams, after all possible life had been obtained from the original installations.

Manganese steel was expensive, and because of shrinkage stresses in the material during the process of casting, it was impractical to make these shells more than 2½ to 2¾-in. in thickness. This meant throwing away a lot of expensive metal when the shell was worn thin in one place. The suggestion was made that the metal commonly known as white iron, might be suitable for these shells. This so-called white iron is very hard, and when properly doctored up to give it the requisite toughness, and properly handled in casting, it was thought that it might give satisfactory service. It had this advantage, that it could be cast in any thickness desired without trouble from shrinkage stresses; also it could be obtained locally at a cost considerably below that of manganese steel.

Some study was given to this matter, and finally two white iron shells were made. They were given a thickness of about 5½ inches at the bottom where the most wear occurs, and the thickness was gradually reduced to about 3½ to 4 inches at the top. Transverse ribs were cast around the shell to prevent it from splitting when it had become worn thin. The first of these castings was very hard, and the finishing work on the few surfaces that required it was done with great difficulty. This shell gave very satisfactory service compared with the others that had been in use up to that time. It finished its first job, and a second one, handling about 560,000 cubic yards of material, and still had 2½-in. thickness of metal left in the bottom. The second of these shells was made somewhat softer than the first. It finished the job where it was first installed and was judged to be not more than half worn out after an output of 430,000 cubic yards. Three other white iron shells were cast later, slightly different compositions being used in each of them, but unfortunately, in the effort to reduce the cost of finishing, toughness and durability were sacrificed, and although one of the later shells gave fairly good service, none of them were as good as either of the first two. It was found also that the metal in these shells, particularly the later ones, was not homogeneous. Two of them wore entirely through at soft spots while only ordinary wear

occurred elsewhere. By the time this much experience had been gained from these shells, the work was getting to the point where there was pumping equipment enough on the job to finish it, so that the experiments were not carried on to conclusion. But there is no doubt that some special metal, based on the white iron formula, is well suited to the manufacture of these dredge pump shells, and will give service comparable with manganese steel almost pound for pound, with the added advantage that the thickness of the castings may be increased at the points where the greatest wear occurs, thereby wasting less metal when the shell is finally worn out, and getting longer life from the shells at a considerable saving in cost.

Incidentally, white iron castings were tried for runners for some of the original dredge pump equipment, but were not satisfactory. As the manganese runners as later used, with removable shoes, were giving good service by that time, no further investigation of white iron runners was made.

Dredge Pipe. The first section of discharge pipe beyond the dredge pump was usually a more or less permanent installation, and for that section standard 15-in, wrought iron flanged pipe was used, having a thickness of ⅜-in. to ½-in. The standard pipe for the rest of the discharge line where frequent moving was necessary, was riveted or welded pipe of No. 8 to No. 10 gage steel plate, built up in 16-ft. lengths, with stove pipe joints. When the installation was first planned, it was expected to use the standard riveted dredge pipe of mild steel. It had an average life of about 100,000 cu. yds. for material such as was handled at most of the dams. In co-operation with the American Rolling Mills Company, of Middletown, a high carbon, electrically welded steel pipe was worked out, which was manufactured at a cost practically the same as that of the standard riveted pipe, and gave service about three times as long. On two of the first installations this high carbon pipe was laid in the discharge lines along with the riveted pipe in alternate lengths, so that the two were subjected to exactly the same service conditions. It very soon became evident that the high carbon pipe was far superior, and it was used exclusively after the first order of mild steel pipe had been worn out. At Germantown, where the material pumped had a preponderance of gravel, the average life of the mild steel pipe was a little under 100,000 cu. yds., while that of the high carbon pipe was about 315,000 cu. yds. At Taylorsville,

the mild steel pipe had a life of about 150,000 cu. yds., while the high carbon pipe averaged about 500,000 cu. yds.

At the joints, the pipe was held together by loops of wire wound around projecting lugs, near the end of each length. To insure an even wear around the circumference it was the practice to turn each length of pipe one-third the way around for each run. This was easily kept track of by making three marks around the circumference 120 degrees apart, numbered 1, 2 and 3. Obviously the lower third of the circumference of a dredge pipe line gets practically all the wear, and this method distributed the wear uniformly in the pipe.

Friction heads in the dredge pipe varied from 4 ft. to 14 ft. per 100 at the different dams, depending upon the kind of material being pumped. The lowest records were at Taylorsville, as would be expected, where the material being handled was glacial till and clay. At Lockington the average friction loss was about 6 ft. per 100, which is lower than at any of the other dams where heavy gravel was being pumped. It will be remembered that there was blue clay in the borrow pit at Lockington which was drawn upon as needed to give a proper mixture of materials for the dam. Apparently the lubricating effect of this blue clay, in the amounts required to give sufficient fines for the hydraulic core, had a marked effect in reducing the friction loss in the discharge pipe. It may be also that the sluicing of all materials to the sumps at Taylorsville and Lockington had something to do with the lower friction losses, because of the thorough working over of the material by the giants and in the ground sluices. At Germantown and Huffman the friction heads averaged around 7 ft. to 10 ft. per 100, and at Englewood at times they ran as high as 10 ft. to 14 ft. The maximum velocity maintained in the dredge pipe was approximately 12 ft. per second, except that at Englewood, where the highest friction losses occurred, the velocities in the dredge pipe were sometimes increased to 14 ft or 15 ft. per second.

These high friction heads had the effect of limiting the lengths of discharge lines and requiring the use of booster pumps at an earlier stage of the work than had been expected. For example, one of these dredge pumps, good for service against a head of 135 ft., would be limited to a discharge line 1000 ft. long, if it were pumping against a static head of 20 ft. and the friction loss in the pipe ran up to 12 ft. per 100. There was no economy in

trying to work the pumps at a disadvantage under such conditions.

Partly because of the high friction losses, and partly to reduce wear in the pipe, it was the general practice to line up the discharge pipes carefully each time they were run out. It was not usually feasible to do this for each length as it was added on, but the line was straightened up as soon as possible after it was put into operation. There is no doubt that this is an economical procedure, both in saving power, and in prolonging the life of the pipe. It also was conducive to smoother operation of the pumping plant, in that clogging of the pipe was less likely to occur than if there were sags in the line where the heavy material would tend to collect and start to form a plug.

Pumping Practice. The first requisite for satisfactory operation of the pumping plant was a reasonably steady flow of materials to the pump. This was accomplished by careful training of the nozzle man in the hog-box. With a little practice he could maintain a steady flow, at the same time working boulders and objectionable lumps of clay off to one side, and preventing the material in the pile from sliding off in masses. It was the general practice for the nozzle man to feed the material to the sump as fast as he could unless he got a signal from the pump man to slow down. An electric signal service was installed between pump man and nozzle man for this purpose.

Given a fairly steady flow to the sump, the next thing is to maintain a steady discharge, by avoiding clogging or plugging in the suction, in the pump, or in the discharge line. It was found that the fluctuation of water level in the sump, together with the readings of vacuum gage, pressure gage, and ammeter, was the most prompt and reliable indication of impending trouble. After the pump speeds and water supply have once been adjusted to the economical discharge from the hog-box, any rise or fall of water surface in the sump may be taken as a danger signal. The gage and ammeter readings help to locate the trouble. For example, if the water in the sump rises, and the reading of the pressure gage increases at the same time that the ammeter reading decreases, it means that the discharge line is being overloaded. On the other hand, if the water level in the sump lowers, and the reading of the pressure gage lowers, while the ammeter indicates increased power consumption, then the pump is not handling as much material as it might. If the water in the sump rises, and

the reading of the vacuum gage increases, with the pressure gage showing decreased pressure, trouble is starting in the suction line. A trained operator can read these signs very quickly and accurately, and therefore, for most efficient service, it was necessary to have all of these indicators within plain sight of the operator. In some cases it was not possible for the operator to see the water level in the sump, and in such cases it was necessary to arrange an electrical device which would register, on the operator's switchboard, fluctuations of water surface in the sump.

Obviously, all the dredge pump motors had to be of the variable speed type. The best plan was to hold these motors, for normal operation, two or three notches below the point of maximum power. Then, when there was indication of an overload in the discharge line, a gradual increase in speed was usually sufficient to clear it out. If the trouble did not clear up immediately, another increase of speed, together with a signal to the nozzle man to slow up the feed from the hog-box, was usually sufficient. If indications showed that there was trouble in the suction line, the feed from the hog-box would be shut off immediately and the trouble could usually be overcome by alternately slowing down and speeding up the pump.

During the first year or two of dredge pump operation, plugs in suction and discharge lines were not uncommon. Even the pumps themselves were sometimes choked up with a nest of large stones, or by a bar, or stick of timber, or something of the sort that would occasionally get into the sump in some unknown manner. As the pump operators became more skilful, however, and the significance of sump levels, together with gage and ammeter readings, became better understood, trouble from plugs became more and more infrequent, until towards the end of the job they were almost entirely eliminated. At the same time average daily outputs were constantly being increased.

Now it is evident that the easiest way to avoid plugs in the pumping system is to continually pump a lean mixture. A pump runner can always play safe by constantly keeping down his percentage of solids. One of the problems of this job was to encourage the pump man to keep his discharge line loaded almost to the point of plugging, without actually reaching that state. The cost of pumping operations was practically the same, per day, whether the percentage of solids was low or high. If a lean mixture was being run, the actual quantity of embankment material

handled during a given period might be less than half what could be obtained without perceptibly increasing the daily cost of operation. Graphic ammeter charts, and records of cars dumped, were posted in conspicuous places and promoted the friendly rivalry necessary to produce best results. As a matter of fact, as will be explained later, the pumps were not crowded up to maximum possible output by the excavating equipment at any of the dams, so it was only necessary to maintain a reasonably steady gait at the pumps, in order to keep up with the delivery of material to the hog-boxes. There was always the chance, however, to keep the power consumption down to a minimum for the amount of material handled, so the ammeter and car records, taken together, gave a basis for judging the efficiency of pump operators.

It has been mentioned that the manganese steel pump runners were equipped with removable shoes. The original idea was entirely one of economy, making it possible to replace worn parts instead of scrapping a whole runner. These removable shoes served another useful purpose, in improving the efficiency of the pumps as the static heads or lengths of discharge lines increased. By having shoes on hand of different lengths, it was possible to increase the effective diameter of the runner by simply putting on a set of longer shoes. (See Fig. 59.) It was the practice to have runners of two sizes, i. e., 36-in. and 42-in. For the 36-in. runner, 36-in., 38-in. and 40-in. shoes were kept in stock. The 42-in. runners could be equipped with either 42-in., 44-in. or 46-in. shoes, so that a dredge pump requiring to be operated under heads varying from say 25 to 150 feet could easily be equipped with any size runner required. This made it possible to continue in economical use, a runner which otherwise would have to be thrown away, or continued in operation at low efficiency. It was quickly found that there is no economy in trying to continue in operation, a runner which is worn down so as to lose its efficiency. The ability to replace shoes quickly, helped to minimize uneconomical operation under these conditions, but it was the practice to go even further, and to discard a runner promptly when it had become worn to the extent that the replacement of shoes would no longer restore its efficiency.

There is a question whether the maximum capacity of these 15-in. dredge pumps was ever determined during the progress of this work. More than 7,000,000 cu. yds. of material were pumped

into the five dams. At none of the dams was the excavating equipment of sufficient capacity to keep the dredge pumps loaded up to their maximum capacity. To show the possibilities of this type of equipment, even with the heavy material that was handled at most of the dams, and to show the gradual increase in output that was attained, the following records are set down; the highest of these, perhaps, approach the maximum capacities of the pumps under these conditions: A run of between 300 and 400 yards per hour for several consecutive hours, or even for full shifts, was not uncommon. At one of the dams 91,500 cu. yds. were put through a single pump in one month of 25 working days, two 10-hour shifts per day. At another dam two dredge pumps handled 180,000 cu. yds. in one month, and it is known that the second pump was in operation only a part of the time.

One pump handled 2100 cu. yds. in 8½ hours of pumping, or an average of 247 cu. yds. per hour.

Another handled 2583 cu. yds. in 9 hours, an average of 287 cu. yds. per hour; the same pump at another time put out 2808 cu. yds. in 9½ hours, or an average of 295 cu. yds. per hour, for practically a full shift.

There is another record of 2916 cu. yds. in 8½ hours, an average of 343 cu. yds. per hour; another of 4000 cu. yds. in 9 hours, or 444 cu. yds. per hour; another of 4050 cu. yds. in 8¾ hours, or 463 cu. yds. per hour, and another of 4590 cu. yds. in 8.8 hours, or 522 cu. yds. per hour.

The highest recorded rates of pumping were 2800 cu. yds. in 4.83 hours, or an average of 575 cu. yds. per hour, and 2781 cu. yds. in 4.41 hours, or 630 cu. yds. per hour.

These quantities are given as in the embankment of the dam, car measurements being corrected to an average of 8½ cu. yds. per car, which is the result of a long time record of carloads as equated against actual measurements.

Booster Pumps. Booster pumps were provided for when the original installations were planned, and their use was similar at the various dams, although their locations in the line were varied. At Lockington the booster pump was set in the main pump house and could be connected up direct with either of the two primary pumps. At the other places the booster was set some distance out on the discharge line. At Germantown and Taylorsville this

distance approximated the reach of the primary pump, while at Huffman the booster was only a short distance up the slope above the main pump house. There was some discussion as to whether the location of the booster, along the discharge line, made any difference in the efficiency of the system. Theoretically, if the material being pumped were a perfect fluid, there would be no difference, but it is very evident that the material handled by dredge pumps, particularly that under consideration here, is not by any means a perfect fluid, and there was a strong opinion among some of the men interested, that there was some definite location on the discharge line where the booster should be installed to give most economical service. No definite information on this point was obtainable. The installations worked satisfactorily at all of the dams, and so many varying conditions were present affecting the efficiency and output of the pumping units, that it was not possible to arrive at a solution of this question.

The boosters were exact duplicates of the primary pumps, and the essential details of their operation were similar. As the discharge line of the primary pump connected into the suction of the booster, the latter was operated normally under some pressure. This varied from a maximum at Lockington, to a minimum at Germantown or Taylorsville, but was an appreciable amount even at the latter places. For best results both pumps should be run at the same speed, and therefore it was necessary for the two operators to be in close communication in order that one might respond promptly to a necessary change of speed on the part of the other. This was accomplished by a simple electrical connection which indicated on the switchboard at each pump how far open the other one was running. This answered the purpose admirably.

Elevated Sumps at Englewood. The use of a booster pump on the line practically doubles the power cost for each cubic yard of material handled. On account of the height and length of the embankment at Englewood, two or more booster pumps would be required on the line for the upper part of the dam, representing a large yardage, if all the material were to be handled through the original hog-box. After some study of this problem, it was concluded that it would be cheaper, at Englewood, to eliminate the necessity for some of the booster pumps, by building additional hog-boxes higher up on the slope of the dam, and properly spaced along the slope. This meant, of course, additional cost of

hauling up the grades to these higher elevations, but it was estimated that the saving in power costs for pumping, would more than make up for the additional cost of hauling in the trains.

Accordingly, three additional sumps were built at Englewood along the upstream slope of the dam, and about equally spaced across the valley. They were so located that the whole dam could be completed without the use of booster pumps at all. Complete pumping equipment was installed at each of these sumps, but in order to avoid the necessity of purchasing additional 500-H.P. motors, it was decided to move the motors back and forth between the two end sumps while the pumping for that lift was being carried on from the middle one. In other words, the practice was to start a lift at one end of the dam, and as soon as the work had passed beyond the range of the sump at that end, move the motors over to the sump at the other end, and have them ready to pick up the work as soon as it got beyond the territory covered by the middle sump. Then the next lift would be doubled back on the preceding one and the process reversed.

These elevated sumps were not built all at once, as the construction program did not make that necessary. Therefore, an opportunity was given to put into effect some improvements in sump design which had been suggested by the operations up to that time, and to give further study and observation with the possibility of incorporating other improvements in the later installations. The final results have been shown in the discussion of sumps in a previous part of this chapter. At Englewood it was even possible to discharge the oversize rock from the revolving screen direct into standard gage cars, without intermediate handling.

Whether or not the elevated sumps at Englewood justified their use in place of booster pumps, (and there is no feeling that they did not), it is certain that they resulted in economy on the hydraulic fill operations as a whole, because of the opportunity they gave of carrying forward the studies towards improved sump and hog-box design. They also permitted the work to go on to completion at Englewood without adding another unit to the plant layout, which always means another possible source of trouble.

Preparation of Foundations. Before the hydraulic fill was started, the foundation for the embankment was prepared by stripping off the top soil, to a depth of a foot or more, in order

to remove objectionable vegetable matter, and large roots were dug out. A cutoff trench was dug along the center line of the dam varying in depth from 5 to 15 or 20 ft. This trench was dug by a dragline machine, and the material excavated from it was thrown over into the downstream portion of the dam, clear of the core section. The cutoff trench was to be filled with core material. It was desirable also that there be a layer of impervious material connected with the impervious core, and extending upstream to form a blanket over the porous sub-strata. At most of the dams, except in the river channel itself, this impervious layer was found to be present, but not always in sufficient thickness. Careful examination was made by means of post-hole diggers, over the whole area upstream from the cutoff trench, to beyond the upstream toe at each dam, and where an impervious overburden was not present to a depth of 3 or 4 ft. the thin places were reinforced with artificial patches of selected material put down in layers and rolled. Thus, a complete impervious blanket was formed over the upstream portion of the foundation. In the river channel sections, an artificial blanket over the whole area was usually required, and in some cases this upstream blanket was extended beyond the limits of the upstream toe of the dam, by diverting and properly confining on those areas the waste from the hydraulic core pool. It was endeavored to keep the shortest line of seepage travel under the dam of such length that its ratio to static head, at any point, would be not less than about 7 to 1.

Handling Materials in the Fill. The plans provided that the width of the impervious core of a dam at any point, should be approximately the same as the height of the dam above that point. Extreme care was taken at the beginning of the embankment construction not to allow the core material to spread out beyond its theoretical limits, as that is a condition that has caused trouble at some hydraulic fill dams in the past. To start the hydraulic fill, dikes were thrown up along the upstream and downstream toes, the dredge pipe was laid parallel and close to the line of the outer slope, and as the fill was being shaped up, the pool, and the core material, were quickly crowded in to the area prescribed for them in the original design. This was sometimes accomplished by throwing the discharge line over towards the middle section of the dam; sometimes a dragline machine was used to shape up the work at the start; teams were

used in some cases; the object being to bring the work into shape as soon as possible at the very beginning, so that the regular hydraulic fill procedure could be instituted at once. Figure 60 shows one of the jobs shaped up ready for the beginning of the regular hydraulic fill.

Low levees were thrown up along the outer edges of the slope, and kept ahead of the hydraulic fill, to turn the flow from the dredge pipe in towards the central core pool. These small levees were simple affairs, sometimes thrown up by dragline machines kept on the dams, sometimes by teams, and sometimes by hand.

FIG. 60—FOUNDATION SHAPED UP READY FOR HYDRAULIC FILL

One of the most useful pieces of equipment for this purpose, was the small back-filling machine illustrated in Fig. 61. The dredge pipe was laid just inside these levees and parallel to them as shown in Fig. 62. The natural deposition of the material forms a gently sloping beach from the line of the dredge pipe to the edge of the core pool, and the width of core is controlled by raising or lowering the water level in the pool. The natural result of the hydraulic fill process is to deposit the coarsest of the material under the discharge end of the pipe and therefore near the outer slope of the dam. The rest of the materials are carried down the beach with the water, the coarser of them dropping along the way until, when the edge of the core pool is reached, the retarding of the flow there causes all but the very

finest of the clay and silt to be deposited along the edge of the pool. Within the pool this clay and silt gradually settles down through the water, forming the impervious core of the dam. The beach assumes a natural slope, which is more or less steep according as the material in the fill is coarser or finer. The slopes of the beaches, with materials used in the construction of

FIG. 61—BACKFILLING MACHINE THROWING UP SLOPE LEVEES FOR HYDRAULIC FILL

This little machine was found very convenient for this purpose. Similar work was done at other dams by dragline machines and by teams. This backfilling machine, however, gave most economical results.

these dams, varied from about 10 to 1 for the coarsest gravel, to about 15 to 1 for the more clayey material, like that at Taylorsville.

The fill was usually brought up in lifts varying in thickness from 2 ft. to 4 ft., the thinner lifts where the beach slopes were less steep, so that the raising of the water surface of the pool to the required height would not run it too far back on the beach,

and thus tend to give too wide a core. These lifts were started at one end of a section of embankment, and carried through continuously to the other end by adding on one length of dredge pipe at a time, filling up the layer to the required height as the work proceeded. Care was taken to maintain the shore line of the pool fairly straight, so as to keep the width of core reasonably uniform. (See Fig. 63). In case the flow down the beach from the dredge pipe was allowed to concentrate, there was liability that it might carry too much material with it, or actually

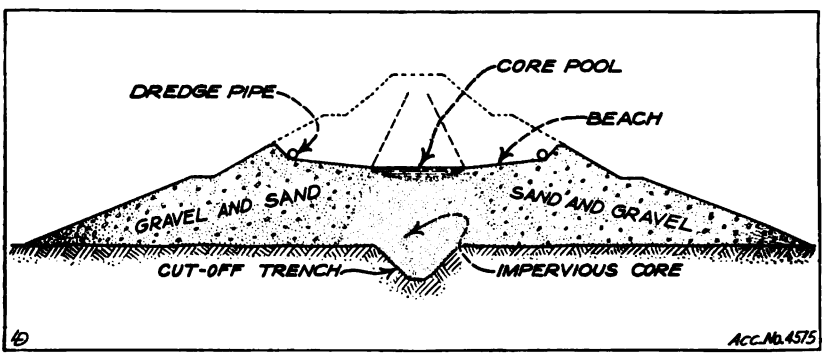


FIG. 62—THE HYDRAULIC FILL PROCESS

Discharge lines from the dredge pumps are laid parallel to the outer slope just inside the slope levee. A natural deposition of material forms a gently sloping beach from the outer slope to the core pool, leaving the coarser material near the outside slopes and carrying the fines to the core section. The width of core is controlled by raising or lowering the water level in the pool. At the edge of the pool the retarding of flow down the beach causes all except the very finest of the material to be deposited, leaving the finest of the clay and silt to gradually settle down through the water, forming the impervious core of the dam.

attain a scouring velocity and take some of the coarse material from the beach out into, and part way across, the core section. These tongues of coarse material shooting out into the core are an ever present menace in hydraulic fill dam construction, the danger being that some of them from opposite sides may meet and form a porous channel through the core section. Various devices were made use of to obviate this danger. Baffle boards on the slope to spread out the flow down the beach, and baffle floats or booms formed by timbers floating on the pool and

moored in close to the shore, were both effective in curing the trouble. A most essential requisite, in this connection, was to keep the core well filled up with solids, by so selecting the materials in the borrow pits that an abundance of core material would be available, and the progress of fill on the beaches would not get far ahead of that in the core itself. Otherwise it was almost impossible to keep the beach material from encroaching on the core.

Deposition of Materials. The theoretical distribution of materials in a hydraulic fill dam, is for the coarsest of the material to be deposited in the outer slopes, with a gradual change to

FIG. 63—HYDRAULIC FILL UNDER CONSTRUCTION
Showing the end of the discharge line from the dredge pumps and the lift of hydraulic fill being brought forward.

smaller and smaller sizes as the edge of the core pool is approached. In the Conservancy dams, while this gradation was discernible in a general way, it was only after close inspection that any difference in character of the material outside the core section, would be noticed. The material near the outside slope did contain a larger percentage of cobbles and coarse gravel, but there was coarse material to be found all through the fill right up to the edge of the core. On the other hand, a close examination would show a preponderance of fine gravel and sand near the edge of the core zone, but sand and fine gravel are also to be found in abundance near the outer slopes; so the gradation, as one follows down the slope of the beach, is not as apparent as

might be supposed. That, in itself, is of little consequence so long as the outer slope material is more porous and free draining than that farther in towards the core, as it undoubtedly is. There is always enough gradation for that. But there is another consideration that should not be overlooked. Just as the fine gravel and sand is trapped in the outside slope material, when theoretically it should be deposited farther down the beach, so a considerable quantity of fine core material is held back in the same way and never reaches the core section at all. This has no effect on the outside slope material, and is in no way objectionable except in cases where there is a scarcity of core material to begin with. Then it simply increases the shortage. In an analysis of borrow pit materials this should be kept in mind. An apparent surplus of core materials should be present in order to take care of this loss in the fill.

Waste-Ways. Some way had to be provided to dispose of the surplus water from the core pool. Except for what was taken care of by evaporation and seepage, all the water that was pumped up into the pool had to be disposed of as soon as it had dropped its load. At the three dams where the combination outlet and spillway structure was used, a semi-circular vertical groove about 4 ft. in diameter was built down the back of the retaining wall, leading to a temporary opening through the wall at the bottom, into the outlet channel. This groove was formed into a well as the fill was brought up, by laying up semi-circular ribs of pre-cast concrete about 6 in. thick, one at a time, to keep pace with the fill and provide a spillway at suitable height for the overflow from the core pool. This provided adequate and safe control at slight expense. When the fill was completed, the outlet from this well was closed with concrete. A manhole cover was placed over the top so that the well might be used in the future for inspection purposes, tapered wooden plugs, which could be removed from the inside, having been provided through some of the rib sections, giving access to the core at various elevations.

At the other two dams pipe outlets had to be provided, and these were extended up as the work on the fill proceeded. It had already been determined by experience elsewhere, that it was practically impossible to maintain these pipe outlets within the core section, because of the fact that there is more or less movement and shifting within the core itself while the filling is going

on, and until the process of consolidation is well along. It was necessary, therefore, to bring up these pipe outlets within the more stable outer sections of the fill, with a temporary structure at the edge of the core pool to regulate the height of water and the outflow. Fig. 64 shows how this was accomplished at Englewood, and this may be considered a typical installation for the purpose. A line of 20-in. double strength vitrified pipe was laid up into the slope from a convenient outlet point, to a point within a few feet of the theoretical edge of the core. There the slope

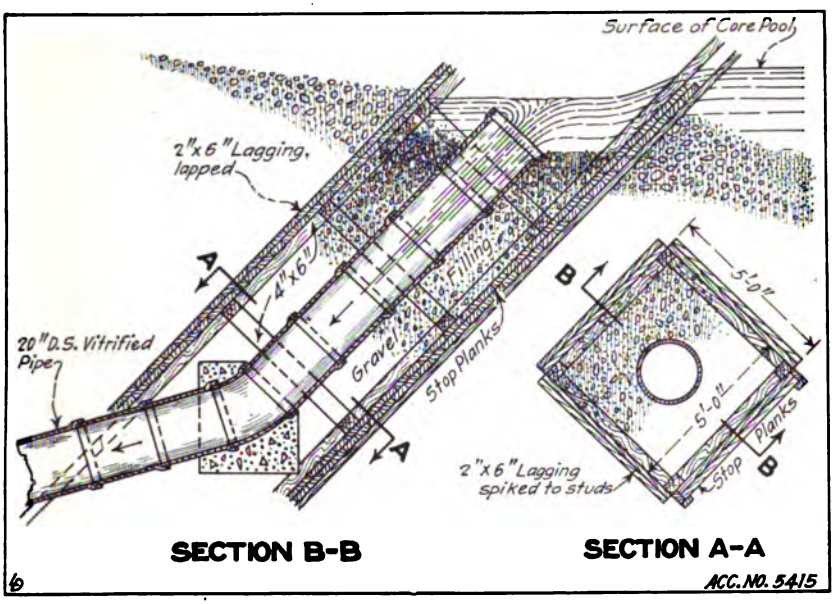


FIG. 64—WASTEWAY FOR CORE POOL
Showing one method of handling the overflow from the core pool.

was changed so as to keep just outside the core limits, and a wooden box was built around the pipe forming a casing or well, on the same vertical slope as the pipe itself. The side of this timber casing facing the core pool was built up of stop planks, which were inserted as the water level in the pool was raised, giving the necessary overflow control. The other three sides were kept above water level. Pipes were added on as the fill was built up, and the space around the pipe within the timber casing was filled with tamped gravel. The pipes were laid care-

fully, with mortar joints, and the bend at the bottom of the steep slope was backed up with concrete so as to prevent settlement or pushing out of place. This was an important feature of the hydraulic fill arrangement and required care in construction to insure continuous service. When the fill was completed, the upper part of the timber casing was removed, and the pipe was filled with fine gravel and left in place in the embankment.

At most of the dams additional waste ways were built at the ends by paving the gutters up the slopes where the fills joined the hillsides, and connecting into the core pools by a length or two of dredge pipe through temporary earth dams, which were removed and rebuilt at higher elevations as the fill was built up.

At Englewood, when the elevated sumps were in use, the waste-ways discharged back into the sumps, in order to save the extra pumping lift for the low pressure water. This had the disadvantage of working over and over the very finest of the core material which was carried back with the waste water, and caused some trouble at times, but on the whole it resulted in economy of operation.

Controlling Width of Core. Several incidental references have already been made to the matter of controlling core widths. First of all, there must be a sufficient proportion of fines in the borrow pit material to give the necessary amount for the theoretical core section, with enough surplus to take care of the amount held back with the coarser materials along the beaches, and enough additional surplus to permit some leeway in the manipulation of the overflow from the core pool. Twenty percent excess over the amount indicated by the theoretical core section is not too much. Without this "working surplus" of core material, encroachment of the coarse material into the core area is inevitable. The core material of the Conservancy dams was practically all fine enough to pass a 100 mesh sieve, and about 80% of it would pass the 200 mesh.

Given the proper percentage of fines in the borrow pit material, the usual method of controlling core width is by raising or lowering the water level in the core pool. Along the edges of the pool, where the flow down the beach is checked by entering the still water, all particles coarser than the very finest of the sands are deposited, and the limits of the core are fixed. The use of shear-boards and floating baffles, to break up concentration of flow down the beaches, has already been mentioned. If tongues

of gravel or sand persist in shooting out into the core, the use of lower lifts in building up the fill will be found advisable. The shore line of the pool should be kept even and regular in order that the core area shall be well defined. A surplus of fines may be wasted, by lowering the spillway at the wasteway. For most satisfactory operation this surplus of fines in the pool itself is almost necessary, furnishing a reserve which can be drawn upon

FIG. 65—HYDRAULIC FILL NEARING COMPLETION

This shows the top of the Englewood Dam. The core section is definitely outlined here, and as the water has been lowered slightly, the appearance of the core material may be noted in the foreground. This is near the top of the dam, and as the core becomes narrower, it is increasingly difficult to maintain it as a definite section. Special methods were used for topping off the dams.

to a greater or less extent as needed, in the regulation of core widths.

This careful regulation of core widths is a most important feature of hydraulic fill dam construction. Once it gets out of control the whole structure of the dam is endangered, either by the formation of porous channels through the dam, or by the sloughing of the outside slope material into the core, thus affecting the stability of the structure.

Core Studies and Tests. Careful study was given to the design of the core for the Conservancy dams, with the result that the core width at any point was made equal to the height of the dam

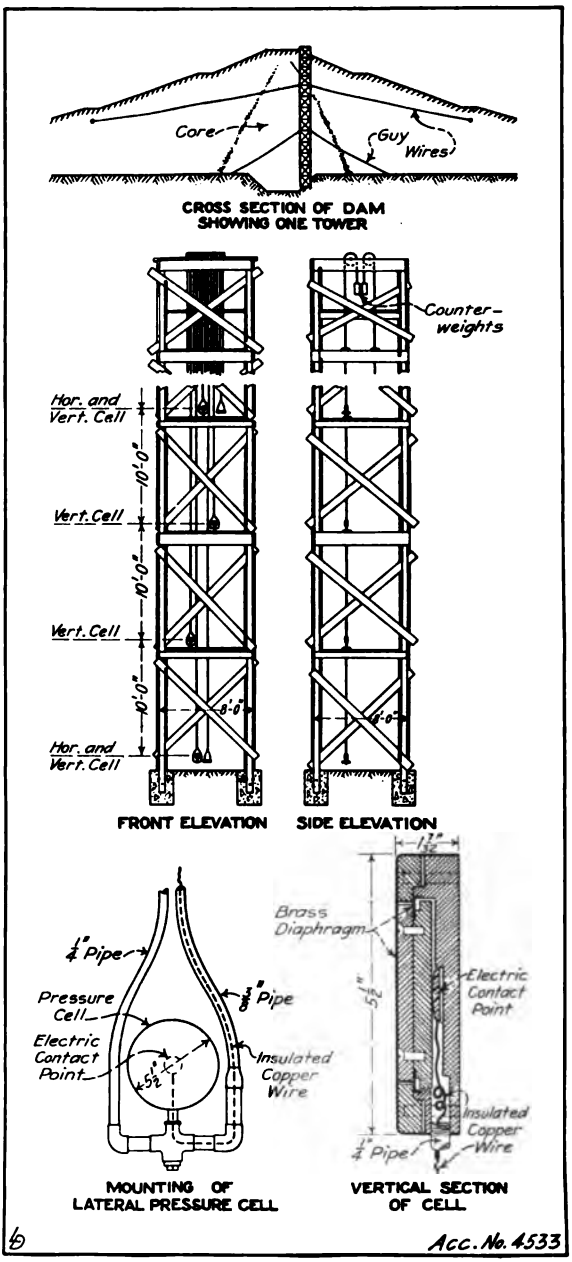


FIG. 66—THE GOLDBECK PRESSURE CELL AND FIRST METHOD OF INSTALLATION

The pressure cell is shown in cross section in the lower right-hand view, and its method of operation will be understood by reference to the text. This method of installation was not satisfactory and was improved later as shown in Fig. 69.

above that point. This gave a core of such shape that sloughing in of the outside slope material would not be encouraged, of sufficient width so that, with reasonable care, the encroaching of sand and gravel too far into the core zone could be prevented, and narrow enough so that additional material to that required to give stable slopes would not be necessary to hold the pressure of the semi-liquid part of the core during construction. (See Fig. 22, Chapter III.)

In order to obtain cores of the dimensions decided on, it became necessary at one of the dams (Taylorsville), to waste a considerable quantity of the core material available in the borrow-pit, and, at two others (Germantown and Huffman), to open auxiliary pits in clay deposits on adjacent hillsides, to make up for deficiency of fines in the main borrow-pits. There are decided advantages in having a surplus of core material available, so that a part at least of the finest of that material may be wasted: First, a better gradation of core material may thus be obtained; in fact, after the selection of the borrow-pits, and such selecticn of material in the pit as is practicable as excavation proceeds, the wasting of fines affords about the only remaining opportunity to control the gradation of the core material; and, second, a more rapid consolidation of core results, by getting rid of some of the extremely fine particles which, otherwise, are held in suspension for a long period, tend to lubricate the whole mass, and retard the process of consolidation by holding up particles which of themselves would settle more rapidly.

In order to study the action of the hydraulic fill cores during construction, it was determined to place at various elevations, in all the dams, a number of Goldbeck pressure cells. Preliminary experiments with these cells at the U. S. Bureau of Public Roads indicated that they would give reasonably correct results if placed in hydraulic fill cores. Briefly, the Goldbeck cell (Fig. 66) is a closed, flat, circular box, 5½ inches in diameter, similar in shape to a shoe-blacking box, the top or bottom of which acts as a movable diaphragm. It is operated by slowly admitting compressed air, through a small pipe, to the inside of the box. The air is supplied from a small tank which may be charged with an ordinary tire pump. The cell is buried at a known elevation in the core, and the movable diaphragm is held down by the pressure of the material. When the pressure of the air on the inside equals the pressure of the material on the outside, the diaphragm

is lifted slightly, an electric contact is broken, and an indicator light in the registering apparatus goes out. At that instant, the gage on the air line indicates the pressure of the material against the movable diaphragm. The operation of the cell requires a movement of the diaphragm of less than 0.0001 inch. The cell may be set either horizontally or vertically, and, thus, either vertical or lateral pressure may be determined.

The limitation of fines in the core material was largely a matter of judgment, based on the analyses of core materials able, preliminary tests made in co-operation with the U. S. Bureau of Public Roads, and analyses of core samples secured from other hydraulic fill dams. After construction was started, frequent analyses of the core materials were obtained through the U. S. Bureau of Soils, and the rates of consolidation were studied. Table XXXI shows the results of some of the analyses of typical core material. Although cores of quite different compositions from those given may show as good results, it may be said that all five of these dams were completed without the least indication of trouble with any of the cores, with ample proof that consolidation has taken place in a thoroughly satisfactory manner, and that the water-tightness of the core, as shown by several tests, is all that could be desired. In other words, while it is not claimed that the gradation shown by these analyses cannot be improved, still these results indicate that cores of about the composition shown are eminently satisfactory from the standpoints of rate of consolidation, degree of consolidation, and water-tightness.

About fifty of the Goldbeck pressure cells were placed in the cores of these five dams. Some have given satisfactory readings, and some have not. In cases where the readings seem unreasonable, or where the cells fail to register, it is sometimes difficult to determine whether the trouble is in the cell or is due to faulty installation. With a few exceptions, there is no evidence that any of the unsatisfactory results have been due to any fault in the cell itself. Defective installation is known to have been the cause of many of the unsatisfactory readings, and improvements in installation have resulted in marked improvements in results. This was a pioneer effort, and, naturally, much was to be learned by experience, before proper methods were determined for meeting the requirements, particularly as to the best methods of installation. The first installation was made at the

Table XXXI.—Mechanical Analyses of Typical Core Samples

Bureau of Soils, Standard Sizing	Germantown			Englewood					Lockington				Taylorsville						Huffman				Average
Percentage of fine gravel, 2 to 1 mm....	0	0	0	0	0	0	0	0	0.3	0	0	0	0	0	0	0	0	0					0
Percentage of coarse sand, 1 to 0.05 mm.	0	0	0	0.4	0	0	0	0	0.9	0	0	0	0	0	0	0	0	0.1	0	0	1.1	0.7	0.1
Percentage of medium sand, 0.5 to 0.25 mm.	14.0	17.3	21.2	0.3	0.2	0	0	0.5	0.7	0	0	0.5	0.1	0	0	0	0	0.2	0	1.8	0.8	0.8	0.3
Percentage of fine sand, 0.25 to 0.10 mm.	1.5	4.2	2.1	4.6	1.6	3.6	0.5	2.6	8.6	1.0	1.5	3.1	1.9	0.5	2.0	0.4	0.1	3.5	2.8	13.7	2.9	6.2	3.0
Percentage of very fine sand, $.10 to 0.05 mm.	0.1	0	0	19.6	16.8	19.3	12.4	5.4	12.6	10.1	33.2	39.0	18.1	19.7	13.9	2.3	4.2	24.6	15.0	25.4	31.4	23.0	18.2
Percentage of silt, 0.05 to 0.005 mm.....	64.2	56.0	55.2	52.7	57.7	55.2	72.0	60.5	52.6	59.3	55.8	40.4	59.7	60.8	61.8	62.9	62.1	50.2	52.5	46.2	50.2	49.2	55.9
Percentage of clay, 0.005 to 0 mm.....	20.3	22.5	21.5	22 6	23.9	22.0	15.2	31.0	24.8	29.3	9.5	16.9	20.2	19.1	22.3	34.3	33.7	21.4	29.8	12.9	13.7	20.4	22.2

Germantown Dam, as shown in Fig. 66. At this dam, the cells were suspended by two ¼-in. pipes, from the top of a framed tower which was built in sections as the construction proceeded. Two pipes were necessary at these installations, one to carry the wire connecting with the registering apparatus and to take the charge of compressed air, and the other to be used as a blow-off

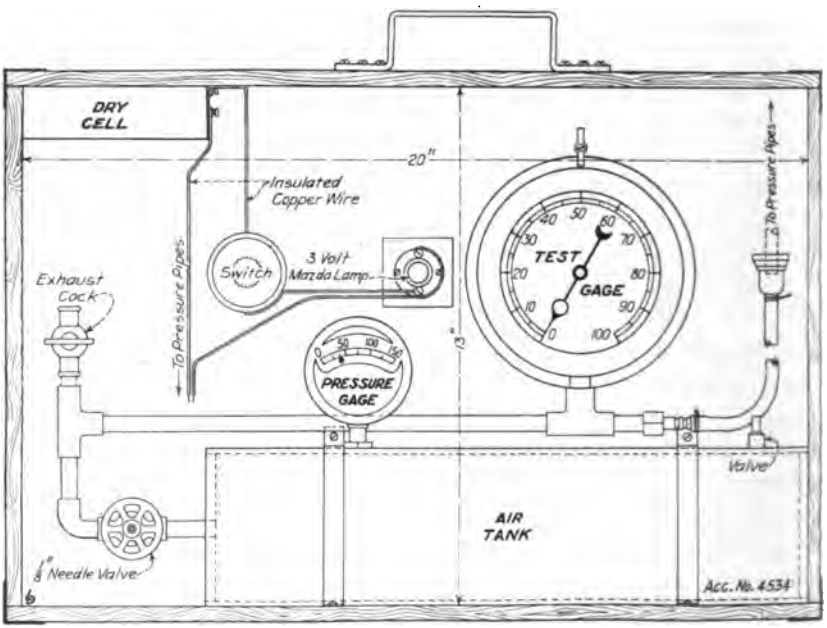


FIG. 67—REGISTERING APPARATUS FOR GOLDBECK PRESSURE CELLS

This apparatus, consisting of compressed air tank, pressure gages, electric battery and light with flexible connections for the pressure pipes, was carried around from place to place in a box not much larger than a suitcase.

in case water entered the pipe or the cell. This method of installation, however, was improved later. (Fig. 69.)

Cells were set in a vertical position (to measure lateral pressures) at 10-ft. vertical intervals, and in horizontal position (to measure vertical pressures) at 30-ft. vertical intervals. As the tower was raised in height, the pipes were extended by connecting another section at the top, and another length of wire

(bell cord) was spliced on and threaded through the one pipe. The tower, surrounded by the soft core, was guyed at intervals to hold it in place.

In taking the readings, a portable gage box (Fig. 67) was taken out to the tower, and connected to one of the pipes from which the cell to be tested was suspended, the pipe being connected to the pressure tank and the wire to the indicator light. A plug was removed from the end of the other pipe and a light

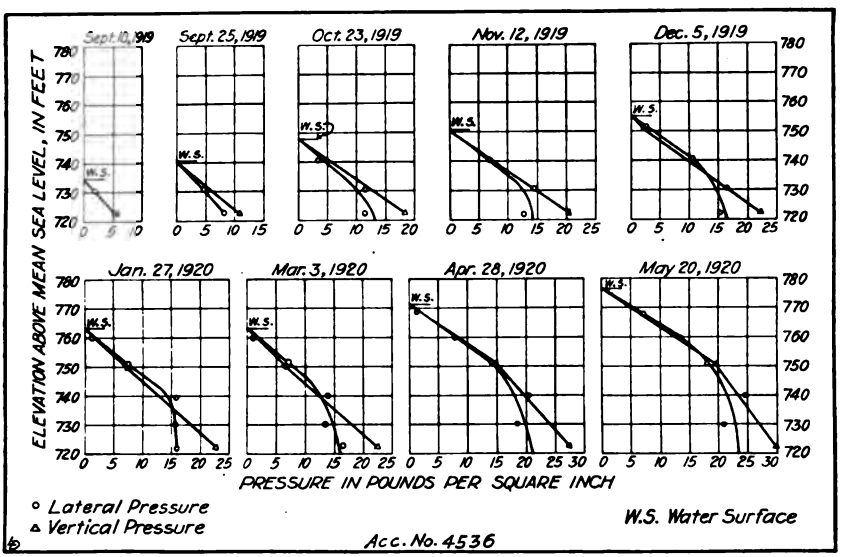


FIG. 68—PRESSURE DIAGRAMS GERMANTOWN CORE
Showing relation between vertical and lateral pressures at various dates and under different conditions of loading. (See Fig. 70.)

"shot" of compressed air was admitted to the first pipe to displace any water which might have accumulated. It was only necessary to force this water out of the way so as to admit air to the cell, since the weight of a water column (if any) in the second pipe would register on the pressure gage, and, therefore, would not affect the correctness of the reading; as a matter of fact, little trouble with water was encountered. The plug was then replaced in the end of the second pipe, and the compressed air was admitted slowly to the cell, until the movement of the dia-

phragm broke contact and put out the light in the gage box. At that instant, the pressure gage indicated the core pressure on the diaphragm. The movement of the diaphragm necessary to break contact (about 0.0001 in.) is so slight that the light may be put out and lighted repeatedly, by proper manipulation of the feed and exhaust valves, without changing the reading of the pressure gage, thus showing that the slight movement of the diaphragm does not disturb the core.

A series of readings at Germantown, covering the period from September 10, 1919, to May 20, 1920, are shown in Fig. 68.

These tests show that after a few weeks the core material begins to consolidate (as indicated by the fact that the lateral pressures are noticeably less than the vertical pressures at the same elevations) and that as time goes on the difference becomes greater, until after a few months the lateral pressures in the lower parts of the core are only about two-thirds to one-half the vertical pressures. This is proof that the core at those points is losing its fluid properties and that consolidation is taking place. These conclusions are confirmed by later examinations of exposed cores, as will be mentioned.

These nine series of readings at Germantown, covering a period of eight months, were very satisfactory. Then things began to happen. The dam had reached a height of from 60 to 70 feet and was being built up at the rate of about 6 or 8 feet per month. It became impossible to obtain further readings on some of the lower cells because air could no longer be forced through the pipe to the cell. This plugging of the pipe was due, in some cases, to the bunching of the insulation on the wire or at the splices, and, in others, to an accumulated deposit of dirt and flakes of galvanizing. When the length of this small pipe is considered (70 to 80 ft. in the case of the lower cells), this latter trouble is not surprising, although it was overlooked at the start. The use of larger pipe, more careful cleaning of the inside of the pipe, and the construction of a trap at the bottom of the pipe to catch the dirt, corrected these troubles in later installations. Some of the pipe joints then began to leak air, and, about that time, the upper part of the tower showed signs of distress, due probably to movement in the upper (partly fluid) portion of the core or to the settlement of material on the guys. This movement of the partly consolidated portion of the core was demonstrated by sinking a cast-iron ball into it, marking its position by

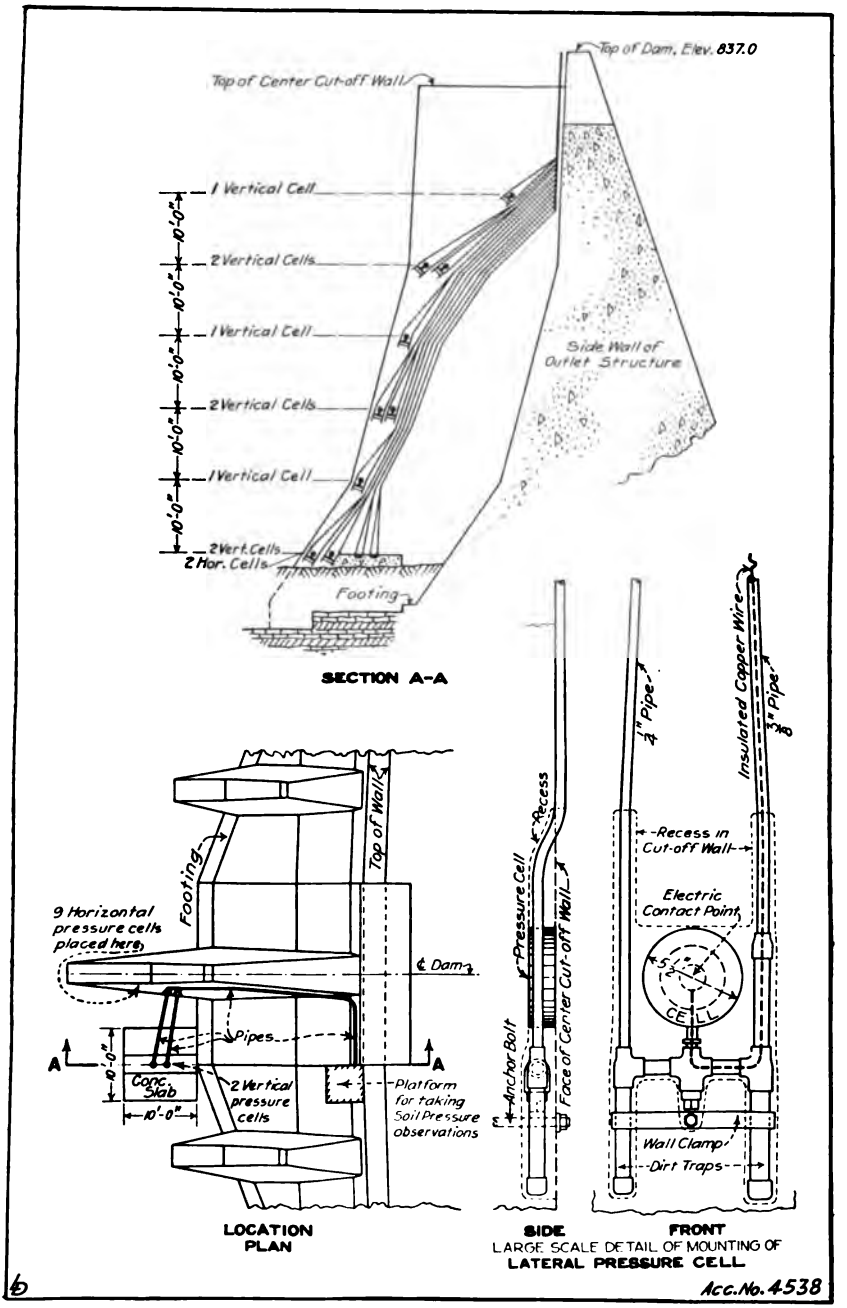


FIG. 69—IMPROVED INSTALLATION GOLDBECK PRESSURE CELLS

This was the installation worked out for Taylorsville Dam after experience at several other places. The cells and pipes were fastened securely to concrete footings or walls, and duplicate cells were installed at several elevations to give a check on the readings. Dirt traps were

a buoy, and checking that position from time to time. This simple test gave positive evidence that, in the earlier stages of consolidation, there is often some movement, or surging, of the semi-fluid core. Such movement, in itself, is of little consequence, except that it must be given consideration in cases where it is desired to construct a permanent tower or shaft within the limits of the core. Difficulties with such towers or shafts have been experienced in other hydraulic fill dams, due no doubt to this core movement. Thus, the pressure cell tests at Germantown, following the period covered by the readings shown in Fig. 68, were of interest, principally, in revealing defects to be corrected in future installations. The Englewood and Lockington installations were too far along to be corrected by the time the Germantown troubles had been analyzed. The readings obtained at these places confirmed the conclusions reached from the Germantown tests, but little additional information was secured.

At Taylorsville, full advantage was taken of the experience gained at the other dams, and a much better installation was made. In this case, the cells were fastened to the side of a concrete cut-off, on the back of a high retaining wall forming a part of the outlet structure. Larger pipes were used, which were fastened securely to the same wall, and instead of joining up the pipe in sections as the dam was raised, the complete installation was made at the start. At the pipe joints, special care was taken to secure a permanently air-tight joint. Dirt traps were provided at the bottom of all the pipes. Fig. 69 shows this improved installation.

Cells in vertical position, to measure lateral pressures, were set at 10-ft. vertical intervals as before, but in this case two cells were set at the bottom and at the 20-ft. intervals, in order to check the readings of one against the other, at those points. Cells to measure the vertical pressures were set at the bottom only, because of the difficulties of making a secure installation in horizontal position at the higher elevations, except at points where the vertical pressure would be affected by friction against the wall. Fig 70 gives the results of readings at Taylorsville to date.

It will be noted that the two cells at the 20-ft. vertical intervals check each other very closely, except at the bottom, where consolidation has taken place to the extent that the lateral pressures are only about 40% of the vertical pressures. This is an indica-

tion that the pressure cell readings are fairly reliable. It might be expected that when consolidation has reached the point where the core is practically a solid, actual lateral pressure might vary at different points to the extent indicated by the readings on the lower cells. The fourth set of readings (November 26, 1921) were taken after the completion of the dam to its full height.

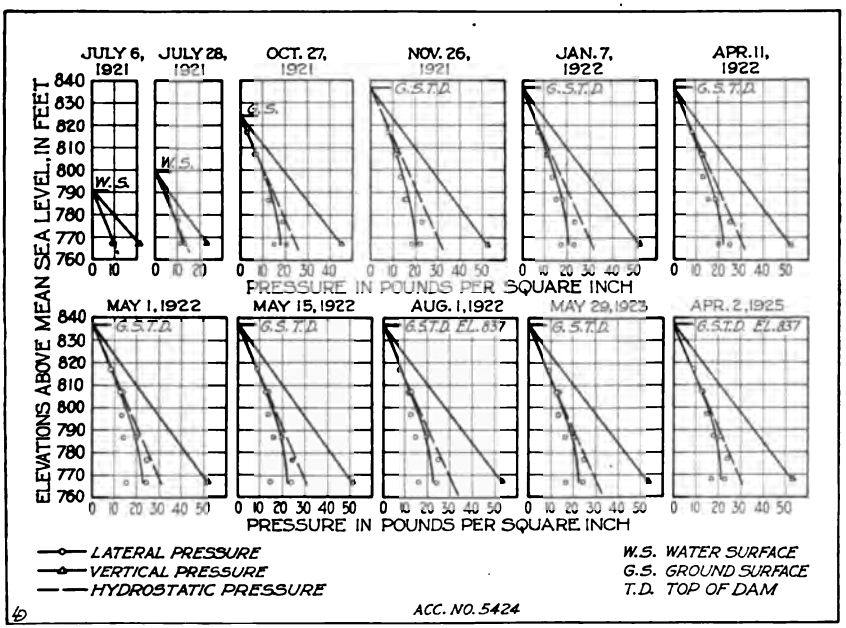


FIG. 70—PRESSURE DIAGRAMS TAYLORSVILLE CORE

Showing relations between vertical, lateral, and hydrostatic pressures at various dates and under different conditions of loading. Note that there is little change in these pressures since the dam was completed 3½ years prior to the last reading, the lateral pressure being about 40% of the vertical pressure and about 70% of hydrostatic pressure for the same depth.

The section of the dam in which the cells were set, was built up from Elevation 767, where the lowest cells are located, to Elevation 837, the top of the dam, during a period of six months; during one month the core was raised 33 feet. It is apparent that the core had reached a state of consolidation on November 26, 1921, beyond which the change is very slight. It may be that after construction is stopped, and no further weight is

added, there will be no change in the pressure on the cells unless the equilibrium is disturbed. Very likely, the condition represented by the last five or six diagrams, where lateral pressures are from 40% to 50% of the vertical pressures, is normal for well consolidated material of that kind. It is not known whether that ratio has been determined for such materials, but 40% to 50% seems reasonable when one considers that for metals it is supposed to be about 30 percent.

The Taylorsville method of installation is apparently satisfactory. The results to date have every indication of being reliable; they check with the readings at Germantown, as shown in Fig. 68, and they appear reasonable in the light of examinations of the cores in place, discussion of which will follow.

It is evident from the data secured, that properly graded core material need not be treated as a perfect fluid even after a few weeks of settlement, and that after it has been in place for a few months, the lateral pressure is only about 50% or less, of the vertical pressure at the same depth. It is believed that this may be accepted as a fact, to be kept in mind in the future design of hydraulic fill dams.

Ball and Rod Tests. Penetration tests with a 6-in. cast-iron ball, and with iron rods, were made at all the dams as the work progressed. The value of the ball test lies in the fact that it has been used at other hydraulic fill dams and therefore it gives the opportunity to make comparisons with cores of other dams, in addition to giving a check on rates of consolidation from day to day. A disadvantage of the ball test is that core material which is only slightly consolidated will support a 6-in. cast-iron ball, so that it never gets down into the more stable material of the core. Several tests were made at the Conservancy Dams to see if the cast-iron ball would continue to settle in the core after it had apparently come to rest. It was found in every case that after the first minute there was no further settlement worth mentioning, even over a period of several days.

Penetration tests with a rod or pipe are valuable in detecting the presence of gravel or sand in the core, or to reveal other encroachments of the coarser materials into the core zone. A rod may be forced much farther down into the core than the cast-iron ball will penetrate, and thus gives opportunity to explore the material which has become well consolidated. Forcing down a rod to refusal gives an opportunity to compare rates of consoli-

dation from day to day. The rod commonly used at the Conservancy Dams was a ½-in. pipe 20 to 30 ft. long, which was operated by two men in a boat. While the same two men, after getting accustomed to this work, would obtain results from day to day that were fairly comparable, it is evident that rod tests on one job cannot very well be compared with those on another, because of the personal equation having such a strong influence on the results. For making comparisons between different jobs, therefore, rod tests are of doubtful value, but as a means of obtaining quick information as to what is going on at any one particular job, they are almost indispensable.

Core Examinations. By far the most satisfactory and conclusive information as to the character of the cores of the Conservancy dams, has been obtained at three of the dams, by examination of the cores where they have been exposed in their undisturbed condition. At the Huffman Dam, the old line of the Erie Railroad passed through the dam site. The construction of the dam was started while the railroad was being relocated, and in order not to delay this construction a cross dam was built along the edge of the railroad right of way, which held back the hydraulic core until after the tracks were moved. The core pool was then drained, the preparation of the foundation was made across the railroad right of way, and the cross-dam through the core section was removed, exposing the end of the core to a depth of 26 feet. Fig. 71 shows the core thus exposed. Although 16 feet of water stood against the exposed face of the core, and although it stood unsupported to a height of 10 feet above the water surface, little difficulty was experienced in cleaning the foundation for the new section of the dam and excavating the continuation of the cut-off trench. The texture of the core in the steep face, and how the bucket load of core, dumped by the dragline machine, retains its shape, may be noted in Fig. 71. The oldest of the core visible in this photograph had been in place about five months, and the top of it had been placed about two weeks before the photograph was taken. It would bear the weight of a man walking across it, after the core pool had been drained. The composition of this core material is represented by the Huffman analysis in Table XXXI. When studying this photograph, it should be kept in mind that the face of the core, below the water surface, was undercut by the excavation for the

removal of the cross-dam and the continuation of the cut-off trench, which explains the slumping near the face.

At the Lockington Dam, when construction had been carried to within about 12 feet of the top, the core pool overflowed, washing a gully in the upstream slope of the dam and exposing the core to a depth of about 30 feet from the top. A similar accident occurred at Englewood, at about the same stage of com-

FIG 71—EXPOSED CORE AT HUFFMAN DAM

Note the compact nature of the material in the freshly exposed face, also how the bucket load of core in the water near the right-hand edge of the picture, holds its shape and shows the marks of the ribs on the bucket.

pletion.* In each case, all the water in the core pool drained through the break. At Englewood, the quantity of water drained was estimated at not less than 800,000 gals. In both cases, the core stood up with almost vertical faces and none of it was washed out, except just at the break. Only about 10% of the material moved by the wash-out was core material. Figs. 72 and 73 show the conditions at these dams immediately after the

*Engineering News-Record, November 3, 1921, p. 744.

FIG 72—EXPOSED CORE AT LOCKINGTON DAM

This washout was caused by an overflow of the core pool. The core material is very compact and stands up in an almost vertical face.

FIG. 73—EXPOSED CORE AT ENGLEWOOD DAM

This was a similar case to that illustrated in Fig. 72. Note that there is no slumping of the core as left by the washout.

wash-outs occurred. It will be noticed in both photographs that the core stands up, unsupported, in almost vertical faces, and that there was no slumping of the core, except just at the wash-out. It is significant also, that there was little or no erosion of the core back from the break on either side, although a large quantity of water flowed over it when the wash-out occurred. (Fig. 74.) Nor was there any appreciable sloughing of the core

FIG. 74—TOP OF LOCKINGTON DAM AFTER OVERFLOW OF CORE POOL

The washout may be seen just beyond the length of pipe laying across the line at the left. Even though the whole core pool ran through this break, there was little or no erosion back from the break on either side.

during the three or four weeks that it stood unsupported while the wash-outs were being repaired. At Englewood (Fig. 73), the top of the core had been in place for less than two weeks before the wash-out. The oldest of the core in sight in the photograph had been in place only about twelve weeks.

Samples of the core material from these places were taken in both cases. At Lockington, five samples were taken at different places, after digging into the exposed face far enough to

reach material that had not been disturbed or affected by the wash-out. Each of the samples was secured by using a 7-in. length of stove pipe, biscuit-cutter fashion, and immediately soldering on a top and bottom, thus forming an hermetically sealed can, full of core as it lay in the dam. The moisture content of these five samples, as determined by the Bureau of Soils, varied from 21.8% to 22.6% by weight. The mechanical analysis averaged, as follows:

1-0.5 mm.	0.5-0.25 mm.	0.25-0.10 mm.	0.10-0.05 mm.	0.05-0.005 mm.	0.005-0 mm.
0	2%	6%	15%	55%	21%

which agrees well with the typical analysis of the Lockington core samples, given in Table XXXI. The low moisture content shows that the core gives up its surplus water within a comparatively short time; there is no doubt of that after an examination of the exposed core in place. Similar samples taken at Englewood had a moisture content of about 24 percent. The weight per cubic foot was about 121 lbs. A large number of pieces of core material at Englewood, some of which were 1 cu. yd. or more in size, were found at the foot of the slope after the wash-out, which showed the toughness of the core material. The Englewood wash-out occurred in a section of the dam that had been built up as much as 30 feet vertically in one month, so it is evident that consolidation takes place rapidly, even when construction progress is fast.

Core Drainage. There is a difference of opinion as to how the surplus water escapes from the core. Experience at the Conservancy dams indicates that this process is more rapid than could occur by ordinary filtration action. It is very likely that some of the water drains out laterally, but there are indications that the greater quantity is displaced and forced up, vertically, by settlement of the solids and by pressure due to the superimposed material. In many places, where pumping had been discontinued temporarily, innumerable little springs might be seen in the bottom of the core pool, where water was being forced up by piping. Fig. 75 is an illustration of this at one of the dams, where the core pool had been drained preparatory to topping the dam.

There is further evidence that comparatively little of the water in the core material escapes laterally. While the dams were being built, seepage water was always in evidence at the toe of the porous slopes. This flow was greatly diminished at times

when the dredge pumps were shut down and the level of the core pool had dropped one or two feet. During the construction of hydraulic fill dams in the arid West, vegetation will grow at the foot of the slopes while hydraulic fill is in progress, but as soon as the work is discontinued, the vegetation will die for lack of water, the explanation being that along the edge of the core pool, immediately below the surface, is a strip of porous slope material not yet silted up by the core; as soon as the water surface drops below this unsilted material, little water escapes through the slopes, and the core pool holds the remainder of the water for a long time. The little springs are still in action, however, and the consolidation of the core goes on, forcing the surplus water to the surface, as illustrated in Fig. 75. It is difficult to explain the rapid consolidation of the Conservancy cores in any other way.

Topping Off the Dam. When a hydraulic fill gets to the point where its width is less than 100 ft., it becomes difficult to maintain the core pool intact, with pumping equipment of the size used here, and this becomes more and more difficult as the width of the fill decreases.

At the Conservancy dams the spillway elevation was from 12 to 16 ft. below the tops of the embankments, and up to about that point the embankments were wide enough so that the core section could be maintained. Shortly after the spillway elevation had been reached, however, it became necessary to modify the regular hydraulic fill routine as far as maintaining a separate core section was concerned. While it was not absolutely essential to maintain a definite impervious section more than a few feet above spillway level, it was desirable nevertheless for the embankment to be reasonably impervious all the way to the top.

The simplest methods of topping off the dams were employed at Taylorsville, because there the material was of such nature that the mixture as it came from the dredge pipe was fairly impervious without any re-sorting. It was only necessary there to keep this top material well mixed as it was laid down, and to control the waste water so that it would not break over the slopes. This latter requires more and more careful watching, of course, as the top of the fill is approached. Instead of using the two discharge lines, one on either side of the fill, one line laid along the center was substituted. The height of lift was reduced somewhat, and the upper layers of the fill were placed by starting at the far end and backing away, taking off one length of pipe

at a time. Low levees were then thrown up along the edges of this new layer, thereby working over the material to some extent and helping to mix it more thoroughly, and another layer was built up in the same way, by starting at the far end and backing away as before. At the very last, extra material thrown out into the levees along the shoulders was used for grading and shaping up for the roadway on top of the dam.

At the other dams where a large surplus of clay was not available, as it was at Taylorsville, the topping off was not quite so

FIG. 75—CORE SPRINGS LOCKINGTON DAM
Showing how the core material gives up its water by vertical piping.

simple. At Germantown, for example, when the point had been reached where it was decided to abandon the maintenance of the separate core section, the pumping was discontinued for a few days while the small dragline machine, which had been used for throwing up the beach levees, excavated a trench to the depth of several feet, out of the core section, along the entire length of the dam. This material was used to build up levees on either side at

the edge of the slope. Care was taken to so mix the material in taking it out, that the levees thrown up in this way would be both stable and impervious, and at the same time the material left in the bottom of the trench was well mixed up, so as to break up any possible strata of pervious material which might have worked its way across the core where it was so narrow.

After this trench had been completed, pumping was started again with borrow pit material selected to yield about 50% clay and 50% gravel. One discharge line was laid along the center, and the trench was filled nearly to the top in this way, starting at the ends of the dam and working toward the center. The dragline machine followed up the dredge pipe as the fill proceeded, and again dug out a trench along the center portion, mixing the material to the extent required and casting it up into levees on either side. Then the center trench was filled again by pumping, and so on until the dam was finally completed.

At Huffman the method was used of raising the upper part of the dam in very low lifts, usually limited to between 6 in. and 12 in. in thickness. This meant that the pipe lines had to be extended or shortened rapidly, and great care had to be exercised in controlling the waste water, so that it would not break over the slopes. Eventually, however, it was impossible any longer to maintain a core section, and the upper 12 ft. of the fill was made by following the methods used at Germantown, except that the final lift was made by working out from the center of the dam, instead of backing in towards the center as was done at Germantown. The dragline machine kept just ahead of the pipe line, reaching out and picking up material from under the pipe discharge, and using it to build up the levees. At the other dams, methods of topping off were similar to these.

The plans provided for a roadway along the top of each of the dams. In finishing off the embankment, it was attempted to leave a surface which would be suitable for use as a roadway until such time as traffic over the dams might warrant the use of hard surfaced roads. In several cases, however, it was found that the material as laid down, which apparently contained sufficient fines to give a fairly impervious mixture, did not have sufficient binder material to make a good road surface. It was necessary in those cases to bring in clay to supply the necessary binder material, and this was spread in thin layers, harrowed into the surface, and rolled, until a satisfactory road surface was

obtained. Necesary gutters were constructed along the sides, and the surface drainage was taken care of either by pipe drains, or open drains, down the slopes, spaced about 400 ft. apart.

Finishing and Planting Slopes. The matter of keeping the slopes trimmed to line and grade was usually taken care of as the fill was brought up. The dragline machine, or the men throwing up the levees, had time to do this as the work proceeded. It was usually necessary to go over the slopes again for a final trimming after the hydraulic fill was entirely completed. The original plans and specifications required that the entire surface be covered with a 12-in. layer of top soil, and seeded to grass in order to protect the slope from erosion. Later studies, however, showed that a satisfactory stand of grass could not be maintained under those conditions, as the slope material in the dams was so free draining that enough moisture would not be held to keep ordinary grass alive. At one of the dams the lower part of the downstream slope was given this top soil covering on which it was possible to maintain a satisfactory sod because of the flat slope. At the other dams, however, the use of top soil on the slopes was given up entirely. At Lockington there were enough cobbles and small boulders in the borrow pit material, so that by raking them out as the fill was built up, a slope covering of clean cobbles approximately a foot in thickness was secured. (See Fig. 76.) This was the most satisfactory of slope treatments, but unfortunately the necessary surplus of cobbles was not available at the other dams. Even at Lockington the upper part of the slope had to be finished in another way.

Germantown was the first dam to be completed, and there many experiments were made with various grasses and shrubs which might be grown under such conditions as were presented by these dam slopes. Strips several feet wide, up one side of the embankment and down the other, were staked out and planted with different things so that the various plantings would be exposed to exactly the same conditions, so as to learn what, if any, difference in treatment might be required on the sunny or the shady side of the slope. Out of these experiments came the conclusion that sweet clover, and Japanese honeysuckle, were the two things that would grow best on these slopes, and furnish the necessary protection against the action of the elements. It has been found, however, that after a year or two of sweet clover, the resulting inoculation of the soil, and the nitrogen supplied,

makes it possible to obtain a good stand of alfalfa, which is a still better slope covering, and eventually will drive the sweet clover out.

On the downstream slopes of the dams the only protection required was against rain wash. Clean gravel slopes, although very porous, were badly eroded by hard beating rains, whereas a slight growth of vegetation seems to give the necessary protection and break up the tendency for water to collect in rivulets and corrugate the slopes.

The upstream slopes were paved for a certain distance up from the bottom. Large floods which would fill the basin to near the top would occur very seldom, and the water surface would remain at these higher elevations only for a few days at a time, so it was decided that while substantial protection against wave wash was required on the lower part of the slope, a good growth of vegetation would give sufficient protection to the upper part. Sweet clover made a satisfactory growth by simply being sown broadcast over the slopes, although it helped considerably to scatter a light covering of top soil over the slopes at the same time the sowing was made. Sweet clover being a biennial must be sown on two successive years. This is now being replaced with alfalfa. Honeysuckle roots were planted at intervals of about 10 ft. and it was found best to surround these roots with a shovelful of top soil at the time they were planted.

It has been mentioned heretofore that the surface drainage from the roadways along the tops of the dams, was carried down the slopes at intervals of about 400 ft. This drainage system also afforded the necessary facilities for draining the slopes themselves. The berms afforded the opportunity to collect the drainage at points along the slopes and lead it to catch-basins or inlets which were connected with the main slope drains. At Germantown and Huffman, where there was sufficient rock available, paved gutters were carried straight down the slope. These open drains had the advantage of being easy to keep clean and of being free from danger of clogging. At Lockington it was only necessary to carry the surface drainage from the road down to the first berm, where the cobble surface of the slopes began. Here it was discharged on to the cobble slope by means of a small apron to spread the flow. Below that point slope drainage was not necessary. At Englewood and Taylorsville pipe drains vary-

ing in size from 6-in. to 10-in. diameter, the larger sizes at the bottom, were laid up the slopes and buried deep enough to afford them protection from breakage and from the effects of frost.

Settlement. Records of the amount of settlement were made at two of the dams as soon as the fills were finished. Stakes were set at regular intervals along the top, in such position as not to be disturbed by frost or traffic. Measured settlement after six months, at Germantown, showed a uniform relation to the height of fill at each point, and it was almost exactly ¼ of

FIG. 76—COBBLE SLOPE COVERING ON LOCKINGTON DAM
A surplus of small boulders and cobbles raked out of the fill were used to cover the slope of the dam below the upper berm to a depth of about one foot.

1% in each case. After eighteen months this settlement amounted to ⅓ of 1%, which also was uniform at each point. The depth of fill at these points varied from 32 to 101 ft. below the stakes. At Huffman the settlement after an eight months' period varied from 1/5 of 1% to ½ of 1%, after a period of seventeen months. It is safe to say that the maximum settlement after completion of a well constructed hydraulic fill dam is not likely to exceed ½ of 1%.

Costs. Following are cost figures showing the itemized costs of hydraulic fill at the five dams. These are costs of the whole

operation, including borrow pit excavation and transportation, which have been discussed as separate operations in Chapter IV, and costs for which have been given there. In each case coal is included under "Supplies," but electric power is shown as a separate item. It will be noted that the labor item at Germantown is higher than at the other dams. This is because of the fact that the overland haul for all materials and supplies at Germantown increased the cost of all work there. Plant depreciation is higher at Germantown also, partly because of this additional cost of hauling of the equipment in to the job and out again, and also because the quantities were smaller at Germantown and the period of work was shorter.

The low power cost at Englewood, as compared with the other jobs, is probably due to the fact that the quantities handled there were so much larger than at the other dams, and there was such a wide distribution and use of electric power on that job that its use could be distributed to much better advantage.

Costs of Hydraulic Fill Embankment

		Unit Cost	Percent of Total
Germantown Dam—795,900 cu. yds.			
Superintendence and Overhead	$ 32,458.78	4.1c	4.8
Labor	313,089.70	39.4	46.7
Supplies	130,006.43	16.3	19.3
Power	79,457.89	10.0	11.8
Plant Depreciation	116,732.16	14.7	17.4
	$671,744.96	84.5c per cu. yd.	
Englewood Dam—3,401,500 cu. yds.			
Superintendence and Overhead	$ 112,354.37	3.3c	4.5
Labor	1,174.868.56	34.6	47.7
Supplies	666,966.77	19.6	27.0
Power	192,322.90	5.7	7.8
Plant Depreciation	320,048.43	9.4	13.0
	$2,466,561.03	72.6c per cu. yd.	
Lockington Dam—795,200 cu. yds.			
Superintendence and Overhead	$ 29,852.61	3.6c	5.0
Labor	241,508.74	30.5	42.2
Supplies	115,784.80	14.6	20.3
Power	122,607.40	15.3	21.2
Plant Depreciation	64,899.75	8.1	11.3
	$574,653.30	72.1c per cu. yd.	

Taylorsville Dam—908,500 cu. yds.

		Unit Cost	Percent of Total
Superintendence and Overhead	$ 34,348.86	3.8c	5.2
Labor	297,819.64	32.8	44.6
Supplies	132,170.25	14.5	19.7
Power	154,784.36	17.0	23.2
Plant Depreciation	49,255.09	5.4	7.3
	$668,378.20	73.5c per cu. yd.	

Huffman Dam—1,349,800 cu. yds.

Superintendence and Overhead	$ 49,922.91	3.7c	5.1
Labor	475,473.66	35.2	48.8
Supplies	162,404.06	12.1	16.8
Power	146,162.96	10.9	15.1
Plant Depreciation	140,833.52	10.2	14.2
	$974,797.11	72.1c per cu. yd.	

During the period of this construction, labor was being paid from 40c to 46c per hour, dragline operators from $200 to $278 per month, locomotive runners 50c to 60c per hour, firemen 40c to 56c per hour, brakemen 40c to 50c per hour, oilers 40c to 56c per hour, pump runners 40c to 47c per hour, monitor men 50c to 65c per hour, all men on an hourly rate being given 11 hours' pay for 10 hours' work (time and a half after 8 hours).

The quantities are based on actual measurements to neat lines of the theoretical section. The costs include trimming and other incidental work, and all field overhead.

Puddled Fills at Dams

At several of the dams there were occasions for building a fill of selected material and well compacted, where hydraulic or semi-hydraulic fill methods were not feasible, and where rolled fill methods were not necessary so long as equally good results could be obtained at less expense. Puddled fills were used in some of these cases with good results. The best example of one class of this work was at Taylorsville where the material was best adapted to the particular methods here described.

Puddled Fill at Taylorsville. In building the first section of the hydraulic fill at Taylorsville, it was necessary to construct a cross dam along the river bank to retain that end of the core. This was started as a rolled fill, but it was so expensive, largely

on account of the many delays because of wet weather, it was decided to use a puddled fill instead, as the material seemed exceptionally well suited to it.

A track had already been provided to bring in the material from the borrow pit, and a small dragline machine was there to distribute the material into the fill as it was dumped from the cars. A small duplex steam pump was used first as a pressure pump, but later the water was obtained from the high pressure monitor line of the hydraulic fill equipment. The material was spread in layers about 5 ft. thick. As each bucketful was dropped into place by the dragline bucket, it was given a "shot" of water under pressure by using a piece of ¾-in. gas pipe on the end of a hose. This pipe was long enough to reach to the bottom of the layer, and the practice was to insert it full length and gradually withdraw it so as to distribute the water through the mass. The process was repeated as many times as necessary to give each bucket load as much water as it would take without becoming too soft. This was entirely a matter of judgment, but after a little experience a man was able to control the operations very successfully. It was even possible in some places, where the dragline machine would not reach the toe of the fill, to load up that part of the material with enough water so that it would actually move out very slowly to the position desired, and more important still, it was so handled that it would stop its movement close to the place desired, and not continue to move in the nature of a slide.

The fill was built up layer by layer in this way at a very considerable saving both in cost and time, as compared to rolled fill, and has served the purpose admirably. There was no indication at any time, either during the period of construction or afterwards, that this fill was not entirely satisfactory.

The material used was a glacial till, with a good percentage of rock fragments and rock flour in it; not a tenacious sticky clay, but a clayey material in which the granular characteristics were prominent. It would absorb water readily, and would allow it to drain out gradually. A dense sticky clay is not suitable for this class of work. This point is emphasized because of the fact that the same type of fill was tried at Englewood, where the material was somewhat similar to that at Taylorsville, but of a less granular nature, and there the operations had to be given up, because when the material was given enough water to compact

it thoroughly, it became too soft, and gave up the excess water so slowly as to render the fill unstable, and prevent it being carried up safely to any considerable height.

The first fill of this sort at Taylorsville contained 35,000 cu-yds. and had a total height of about 42 ft. There was no sliding or sloughing at the bottom to cause any apprehension. Two tests were made during the progress of the work which proved very satisfactory. A test pit was dug to a depth of about 5 ft. The walls of this pit stood vertical without sign of flaw or break, the material in the walls being solid, smooth in texture, and thoroughly compacted. At another time a railway cut for one of the temporary tracks was excavated across one end of the fill to a depth of 8 ft. on one side, with a side slope of about ½ to 1. The character of the material was solid and firm, well compacted, and this cut stood open for several months during construction without a sign of movement in the walls. Another indication of the solidity of the embankment was that the dragline machine, which was a 70-ft. boom machine, carrying a 2-yard bucket, with a working weight of about 62½ tons, rode upon this structure during the time it was being constructed without any difficulties from settlement.

It became necessary at Taylorsville to put in several other cross dams at various places, most of which were built by this method. Also the portion of the dam east of the outlet structure was put in by this method as it was not of sufficient amount to warrant the use of hydraulic fill methods. This material was brought in and distributed by wagons, otherwise the methods were the same as those described.

Costs. Representative cost figures, on 43,600 cu. yds. of this kind of work, are given below:

Cost of Puddled Fill at Taylorsville Dam

		Unit Cost	Percent of Total
Superintendence and Overhead	$ 791.84	1.8c	4.3
Labor	9,481.21	21.8	52.4
Supplies (Including electric power and coal)	6,778.37	15.5	37.3
Plant Depreciation	1,117.00	2.5	6.0
	$18,168.42	41.6c per cu. yd.	

Labor was paid 40c per hour, dragline operators $225 to $250 per month, locomotive runners 55c per hour, firemen 45c per

hour, men on an hourly rate being given 11 hours' pay for 10 hours' work.

Puddle Fill at Lockington. The west end of the dam at Lockington, beyond the river channel, was a long low fill. The hydraulic fill section of the dam stopped at the point where this fill was about 26 ft. in height. The type of construction beyond that point was a dragline fill with a puddled core.

The material was obtained from a borrow pit along the upstream side of the dam, a wide berm being left between the edge of the borrow pit and the toe of the fill. A dragline machine with 100-ft. boom and 2-½-yd. bucket handled all the material with one throw. The material in the borrow pit was a sandy loam having a depth of several feet, below which was clay. In order to get material for the puddled core, water was pumped up from the river and the borrow pit was flooded; the clayey material taken from the bottom of the pit and worked up with the water where necessary, was deposited in the core section of the fill, while the outside slopes were built from the dry loamy material taken from the sides of the pit.

This gave a fill which was sufficient for the purpose, inasmuch as the head against it would never be more than a few feet; and since it would take a maximum flood to put water against it at all, it was fair to assume that it would have several years to settle before such a flood occurred.

Ample overfill was provided to take care of settlement. After the fill had been in place for eighteen months, and before the final grading was done, settlement records were obtained. It was found that along the center line of the fill where the depth was from 10 to 26 ft. the settlement ranged between 4.7% and 14.8% of the depth of the fill. The average at 10 points was 7.85%.

Costs. The total quantity of material placed in this manner at Lockington was 83,800 cu. yds. The cost per cubic yard was 30.8c as shown by the figures given below. At this time labor was being paid 40c per hour, dragline operators $250 per month, oilers 45c per hour; men on an hourly rate being given 11 hours' pay for 10 hours' work.

Cost of Puddled Fill at Lockington

		Unit Cost	Percent of Total
Superintendence and Overhead	$ 1,550	0.018	5.8
Labor	9,277	0.111	36.0

		Unit Cost	Percent of Total
Supplies (Including power)	4,215	0.050	16.2
Plant Depreciation	10,812	0.129	42.0
	$25,854	0.308 per cu. yd.	

Rolled Fill

Methods of handling rolled fill are well standardized, and there was nothing of special interest about this type of work as used in the construction of the Conservancy Dams, except, that it is interesting to compare the cost of rolled fill with that of the other types being put in at the same time.

At two or three of the dams, rolled fill in fairly large quantities was used for one purpose or another. The material was usually obtained from the main borrow pit and brought in with cars. This material, of course, had to be especially selected for the purpose, but generally speaking that did not add greatly to the cost. At Englewood, where a large portion of the rolled fill was used in cross dams, it was re-handled and placed in the fill by a dragline machine. At most places teams were used for spreading, and in some cases for re-handling. The material was spread in 6-in. layers, and if not sufficiently damp, was sprinkled by hose or watering wagon. Both steam rollers and traction engines were used for rolling.

One of the factors which had a definite effect on the cost of this type of work was that it was interrupted so much in the course of a working season by rainy weather. Not only would the work have to be shut down during a heavy rain, but it was necessary to continue the shut down for a day or two after the rain had stopped, until the material had dried out sufficiently so that the spreading and rolling could be resumed. Work on both the hydraulic and the puddled fills could be continued without interruption regardless of rainy weather, but in this part of the country the necessary interruptions to a rolled fill on this account must be taken into consideration.

Following are cost figures on more than 100,000 cu. yds. of rolled fill, which were placed at Englewood, Taylorsville and Huffman dams. The cost of coal is included under "Supplies." At Englewood an electric dragline machine was used to place a large part of the material in the fill from where the cars dumped it, and the cost of electric power is shown as an item.

At the other dams no electric power was used on this item of work. At Taylorsville a steam dragline and a steam roller were used, making the cost of supplies high, as that item includes the cost of coal. At Huffman most of this work was done with teams, some by contract, making the "Labor" item high, and the "Supplies" and "Depreciation" items low.

Costs of Rolled Embankment
At Englewood, Taylorsville and Huffman Dams

Englewood, Dam—93,900 cu. yds.

		Unit Cost	Percent of Total
Superintendence and Overhead	$ 4,467.91	4.8c	5.0
Labor	57,466.22	61.2	63.3
Supplies	9,974.83	10.6	11.0
Power	2,281.92	2.4	2.5
Plant Depreciation	16,502.14	17.6	18.2
	$90,693.02	96.6c per cu. yd.	

Taylorsville Dam—15,200 cu. yds.

		Unit Cost	Percent of Total
Superintendence and Overhead	$ 638.00	4.2c	4.1
Labor	6,826.60	44.9	43.5
Supplies	7,935.05	52.0	50.4
Plant Depreciation	310.00	2.1	2.0
	$15,709.65	$1.032 per cu. yd.	

Huffman Dam—14,450 cu. yds.

		Unit Cost	Percent of Total
Superintendence and Overhead	$ 605.29	4.1c	4.3
Labor	12,534.31	86.8	90.9
Supplies	551.12	3.8	4.0
Plant Depreciation	122.64	0.8	0.8
	$13,813.36	95.5c per cu. yd.	

Wages and general labor conditions were the same when this work was being done as when the hydraulic and puddled fills were being placed, and while all these cost figures are high, due to the high prices prevailing at the time the work was done, it is believed that the costs given in this chapter afford a fair basis for comparison as to relative costs of these types of fill.

Levee Embankment

Embankments for levees, according to the specifications governing the work, must be carried up in approximately horizontal

layers, extending entirely through the fill. Where the material was deposited in a wet condition, the layers could be not more than 3 feet in thickness, but where the material was deposited in other than a wet condition the layers could not exceed one foot in thickness and some means satisfactory to the engineer must be used for compacting. The object was to secure a compact, stable, and reasonably impervious embankment.

The construction of levees was very often tied in with channel excavation, and construction methods and progress were so interdependent that only a limited amount of data could be recorded on levee construction itself. In many of these cases also, costs of levee embankment were not determinable, as the levee work and the channel excavation were really one job and costs could not be separated.

Some of the levees were built from side borrow. Many of these were built by contract, as that seemed to be the most economical way to handle that particular class of work, and while the actual costs of work are not known in those cases, some of the contract jobs will be discussed and the contract prices will be given.

In some cases the levee work consisted in the construction of new levees, while in many other cases the work required was simply the raising of existing levees to an additional height. In the latter cases the work was very difficult and correspondingly expensive, as it was spread out over long distances with comparatively small yardage per lineal foot; the work involved long hauls; and the interference from trees, which usually line the banks of the old levees and must be preserved when possible, added to the difficulties of the job.

The most extensive work of this kind was at Dayton, where many miles of existing levees were raised to additional heights of 2 or 3 ft. Most of this work was done by a small caterpillar dragline machine with a 60-ft. boom and a 1-½-yd. bucket, working along the foot of the slope, loading the material into 4-yd., 36-in. gage dump cars, which were run on a track on top of the levee, and hauled by 6-ton gasoline locomotives. In loading this material it was necessary to select open spaces between trees, or points where sufficient clearing could be done without injury to the large trees. Usually the cars could be loaded on top of the levees, but in some cases it was necessary to load in the river bed and haul out on a grade. Where the hauls were short, teams

were used instead of cars. It is not possible to give any reliable costs on this kind of work as it was always a part of the channel excavation, and separate costs for the levee work alone were not obtained.

FIG. 77—CONSTRUCTING LEVEE AT FRANKLIN
Material was hauled in trains of 4-yd. dump cars by 18-ton dinkey locomotives. Track was built on trestles of two-post bents, 16 feet apart.

At Franklin is a good example of levee construction by dragline, cars, and trestle. Here the excavation was from a projecting point in the river bank with deep cutting, trimming

slopes, and interference from large stumps, which made the excavation costs considerably higher than they would have been from a favorable borrow pit. The levee was about 5,000 ft. long, had a maximum height of 13 ft. and was made 16 ft. wide on top so as to accommodate a double track, 36-in. gage. The excavation was made by a steam dragline machine with 75-ft. boom and 1-½-yd bucket. The material was hauled in trains of 8 or 10 4-yd. dump cars, over 36-in. gage track, by 18-ton dinkey locomotives. A trestle was necessary the whole length of the fill. This was built of two post bents, 16 ft. apart, with 6-in. x 8-in. caps and 12-in. x 12-in. stringers. Footing blocks were used in place of mud sills, in order to avoid the necessity of leaving timbers lying crosswise in the levee. The caps and stringers and as much of the bracing as possible were removed before the completion of the fill. Fig. 77 shows this levee under construction, with the trestle construction in the foreground, beyond which is the train dumping material into the fill.

There were 70,000 cu. yds. in this levee. It was built at a cost of 83.8c per cu. yd. and the unit costs given below include the cost of excavation, transportation, trestle construction, plant depreciation, and all other expense properly chargeable to the work. The average haul was about 3,200 ft. Wages during the period of construction were about as follows: Labor 40c per hour, dragline operators $250 per month, locomotive runners 60c per hour, firemen 56c per hour, brakemen 46c per hour.

Unit Costs of Levee Construction at Franklin

	Unit Cost	Percent of Total
Superintendence and Overhead	$0.07	8.4
Labor	.393	46.9
Supplies	.197	23.5
Plant Depreciation	.178	21.2
	$0.838 per cu. yd.	

One of the levees at Troy, which was built by contract, had a length of about 3,000 ft.; it followed an old ditch bank which was raised and widened to a top width of 8 ft., with side slopes of 2 to 1. The maximum fill was about 15 ft. The material required to complete the work amounted to 38,000 cu. yds., all of which was obtained from borrow pits, with an average haul of about 850 feet. This work was done with a ¾-yd. revolving

shovel and nine or ten teams. The contract price was 75c per cu. yd.

Another levee at Troy, containing 6,400 cu. yds., was built with 4 slip scraper teams, from side borrow. It was necessary, in this particular case, to use a wide shallow borrow pit. The contract price was 50c per cu. yd., including the surface dressing and seeding.

One of the channel excavation contracts at Troy contained an item of about 60,000 cu. yds. of levee embankment. This work was done with a dragline machine equipped with a 60-ft. boom and a 2-yd. bucket. The material was taken from borrow pits alongside, and practically all of it was put into place without rehandling. The trimming and sloping was done by hand. This was a variable fee contract, by the terms of which the contractor's fee depended on the actual cost of the work as compared to an agreed estimate, hence the actual cost of the work is available in this case, even though it was contract work. The estimated unit price, agreed to by the District and the contractor, was 38c per cu. yd. The actual cost was 27c per cu. yd., plus the contractor's fee and bonus, 8c per cu. yd. in this case, making the total cost to the District 35c per cu. yd.

One of the levees at Hamilton containing 65,000 cu. yds. was built by a dragline machine, under favorable conditions, from side borrow. The levee was about 4,500 feet long, from 8 to 13 feet high, with a top width of 8 ft. and 2 to 1 side slopes. A berm 15 ft. wide was left between toe of levee and edge of borrow pit. The material was top soil and gravel and all of it was placed with one throw. The dragline machine was equipped with a 60-ft. boom and 2-yd. bucket. Following are the cost figures on this work. These figures include the cost of moving the dragline machine on to the work, a distance of nearly two miles, and crossing a canal, requiring about 10 days in all.

Cost of Ford Levee—Hamilton
Total Quantity 65,200 cu. yds.

		Unit Cost	Percent of Total
Superintendence and Overhead	$ 800	$0.012	6.5
Labor	8,014	0.123	66.8
Coal	1,050	0.016	8.7
Other Supplies	825	0.013	7.1
Plant Depreciation	1,323	0.020	10.9
	$12,012	$0.184	

Wages, at the time the work was done were for labor 38c per hour, dragline runners $50.50 per week, firemen 45c per hour, team and driver 80c per hour. Coal cost $3.50 per ton on the railroad siding.

On the east side of the river at Hamilton, south of the Main-High Street Bridge, was a piece of levee about 3,200 feet long, and containing 112,000 cu. yds., built entirely from channel ex-

FIG. 78—DRAGLINE BUILDING LEVEE FROM CHANNEL EXCAVATION

Most of the material was placed with one throw, using an electric dragline with 100-ft. boom and 4½-yd. bucket. The greatest height of this levee was 28 feet above toe of slope on the river side. Material was river gravel.

cavation. The greatest height of the levee was 28 feet above the toe of the slope on the river side. It was built with a 100-ft. boom electric dragline machine carrying a 4½-yd. bucket. For nearly two-thirds of the distance the material could be placed with one throw, but for the remainder it was necessary to handle the material two or three times to get it from the excavation to its place in the levee. In some places it was necessary to excavate below grade to get enough material for the levee opposite.

It happens that in this case the reverse condition, i. e., having a surplus of excavation in places, did not occur. In such case the remedy would be to widen the levee in those places, or to pass the surplus material up or down stream a short distance to where there might be a deficiency.

Where the levee could be built with one throw the operation was simple. The material was river gravel, mostly wet, easy to handle, and easy to compact. The dry material was mixed with the wet, in the fill, in such manner that all of it was sufficiently wet to compact readily. The levee was completed to full height, as the machine moved forward, the section within reach of the machine, after each move, being brought up in layers. Fig. 78 illustrates this work.

Where the material had to be moved more than once, the excavation was thrown up in a windrow as far over as the boom would reach, and on the return trip the machine either put the material into place in the levee, or moved it over again that much nearer to its final position. It was estimated roughly that material could be moved three times by a dragline, in this way, more cheaply than by cars or wagons. Where more than three moves were necessary, it was worth while to figure on other methods of transportation. Fig. 79 shows the machine just finishing a second throw of material into place. The material in the foreground is to be moved again. That opposite the machine is in place in the levee but not shaped or trimmed. Beyond the machine, a piece of finished levee may be seen. An average day's work, on the one throw material with this machine, was 1,550 cu. yds. in two 10-hr. shifts. On the two-throw work, including what little there was that had to be moved three times, the machine averaged 1,200 cu. yds. per 20-hr. day, levee measurement. Separate costs are not available on this work.

On the west side of the river at Hamilton, above the High-Main Street bridge, is a levee about 2,000 ft. long, with a maximum height of 26 ft., which was built from channel excavation by a steam dragline machine with 60-ft. boom and 2-yd. bucket. The material in this levee was practically all gravel, part wet and part dry, and had to be handled from one to three times, some of it being taken from as far out as the middle of the river. This really was work for a larger machine, but none was available at the time. The total quantity, measured in the levee, was 46,000 cu. yds. It averaged at least two-throw work. An average

day's work was 580 cu. yds. in place in the levee, for two shifts of 10 hours each. Separate costs cannot be given, as this work also, was a part of the channel excavation.

A road fill was built in connection with the Dayton work, which, while not a levee job, was so similar that a description of it is included here. This was an approach to a highway bridge and was built from side borrow by a Lidgerwood electric dragline machine equipped with 100-ft. boom and 4-yd. bucket. The

FIG. 79—DRAGLINE MAKING SECOND THROW OF MATERIAL INTO LEVEE EMBANKMENT

The material in the foreground is to be moved again. That opposite the machine is in place but not shaped or trimmed.

length of the fill was 2150 feet, the depth varied from 0 to 24 feet, top width was 30 feet with 1½ to 1 side slopes, and the total quantity was 26,000 cubic yards, all of which was one-throw work. The material was sand, gravel and loam, and all of it was taken from a borrow pit along one side of the fill.

The unit cost of this work was $0.144 per cubic yard, and the machine was working on it for 52 shifts of 10 hours each.

Cost of 26,000 Cubic Yards of Road Fill at Dayton

		Unit Cost
Superintendence and Overhead	$ 140.00	$0.005
Labor	2,233.04	.085
Supplies	175.36	.007
Electric Power (Est.)	400.00	.016
Plant Depreciation	800.00	.031
	$3,748.40	$0.144 per cu. yd.

At this time laborers were being paid 40c to 46c per hour, dragline runners $275 per month, oilers 56c per hour, all men on an hourly basis being given 11 hours' time for 10 hours' work (time and a half after 8 hours).

On several of the levees at Hamilton stakes were set to record settlement. In one place, where the levee, 20 to 25 ft. high, was built by dragline machine from wet river gravel dumped directly into place, the average settlement after six years was only ¼ of 1%. Another levee built of wet gravel but containing large lumps of clay, showed a settlement of 1.5% after six years, due probably to the breaking down of the clay lumps. A levee about 12 ft. high built by dragline of dry loam and gravel showed an average settlement of 4.3% after two years.

CHAPTER VI.—CONCRETE CONSTRUCTION—METHODS AND COSTS

It will be recalled that the outlet structures at the dams are of two types, i. e., the twin conduits at Germantown and Englewood, extending through the base of the dam from toe to toe, and the combination outlet and spillway structures at Lockington, Taylorsville and Huffman, consisting of massive retaining walls facing each other, forming a gap through the embankment, with a spillway weir across the opening between the walls, and the outlet conduits through the base of the weir. At Germantown and Englewood, where the twin conduits were used ,the spillways were separate structures.

Because of the difference in types of these structures, the character of work, as well as construction methods, differed also; but, as would be expected, at the dams where the structures were of similar design, there was also a similarity in the methods used. At Germantown and Englewood, for example, methods and conditions were so similar that a description of one job almost answers for both.

Outlet Conduits at Germantown and Englewood Dams

Description—The outlets through the Germantown Dam consist of two parallel and adjacent conduits 546 feet long, from headwall to headwall. Their cross section is of modified horseshoe shape, 13 ft. wide and 9 ft. high; at Englewood the conduits are 709 feet long, 13 ft. wide and 10½ ft. high. But for temporary use in controlling floods during construction of the dam, these conduits were made much deeper, as is illustrated in Fig. 80. The right-hand section is the temporary size. After construction had proceeded to the point where floods could be controlled by the permanent openings, the lower parts of the conduits were filled in and floored over, as illustrated in the section at the left. These conduits were set in a trench cut out of the rock and trimmed to fit the requirements of the concrete structure. Concrete collars, at intervals around the outside of the conduits, were

put there primarily to prevent seepage through the dam along the line of the structure. At the inlet end of the conduits is a gradually narrowing channel, with concrete walls and floor, and at the downstream end is the stilling pool (for taking up the destructive energy of the water discharging from the conduits

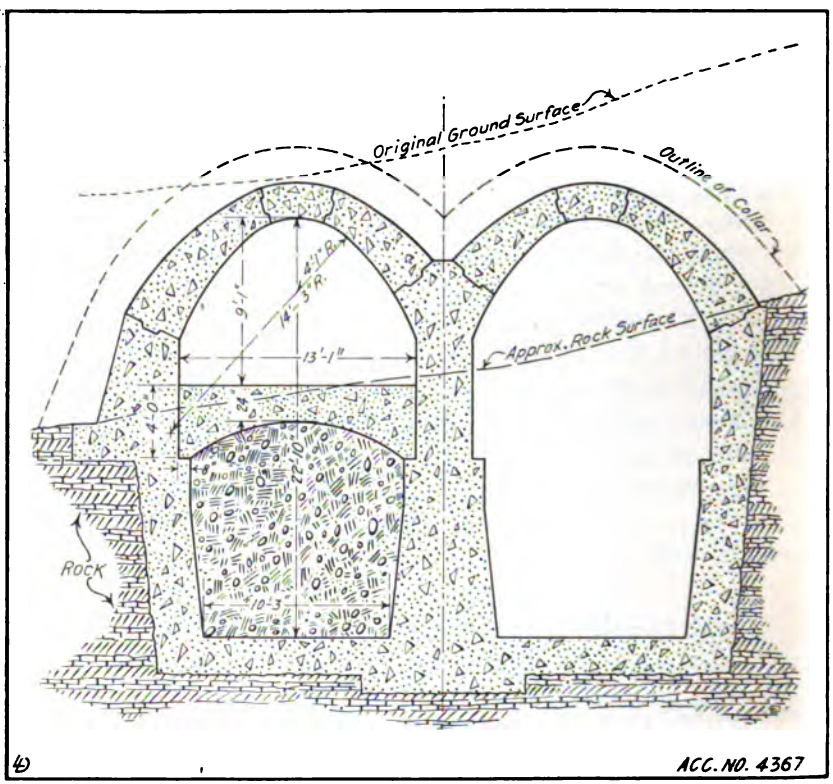


FIG. 80—CROSS SECTION OF OUTLET CONDUITS AT GERMANTOWN

The right-hand section is the temporary size. After construction had proceeded to the point where floods could be controlled by the permanent openings, the lower parts of the conduits were filled in and floored over, as shown in the section at the left.

under high heads) which is illustrated in Fig. 81. By depressing the floor beyond the outlet end of the conduits and placing a weir across the waterway, a short distance downstream, a pool is formed which has the effect of stabilizing the hydraulic jump, or

standing wave, and causing it to form at nearly a pre-determined point, for any possible flood stages. This makes it possible to protect the channel bed from erosion in an economical way by means of a suitable structure as is shown in this illustration. A general plan of this structure is shown in Fig. 23, Chapter III. The features of design of the outlet conduits, which are of more than usual interest, are discussed in Part VII* of these Technical Reports, and the action of the hydraulic jump is analyzed in Part III.† There was one of these stilling pools at the discharge end of the conduits at each of the dams, similar to one another in general design, but with the important dimensions fixed by the requirements and conditions at each place.

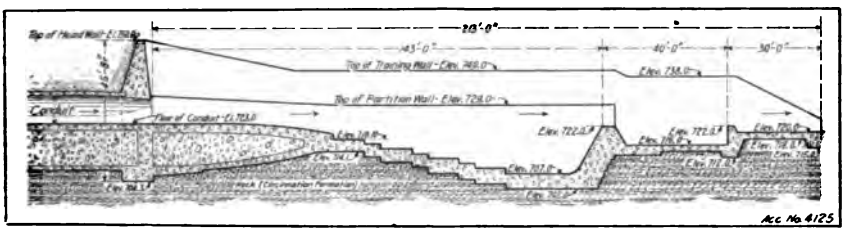


FIG. 81—LONGITUDINAL SECTION OF STILLING POOL

Because of the varying depth of the earth fill upon them between the toes of the dam and the center, the conduit arches vary in thickness at the crown from 15 inches at the ends of the conduits to 2 ft. 6 inches in the middle. Contraction joints were placed at 30-ft. intervals, to take care of temperature stresses longitudinally, and in pouring the arches, joints were formed as indicated in Fig 80 and Fig. 82 to take care of shrinkage stresses, temperature changes, and load stresses in the arches themselves.

Screening and Washing. The plant for handling this concrete has already been described in Chapter III, and Fig. 25 (Chapter III) shows the general arrangement of the screening, washing and mixing plants. The sand and gravel, at all the dams, were obtained at the dam site, and were brought to the plant in dump cars. There was nothing of special interest about the screening and washing operations at Germantown and Englewood except

* Part VII, Technical Reports—Hydraulics of the Miami Flood Control Projects.
† Part III, Technical Reports—Hydraulic Jump and Backwater Curves.

that at all of the plants there was trouble from too much water being carried into the sand bins. The equipment that came with the plant, for removing the excess water from the sand, was a receptacle in the shape of an inverted cone with an automatic counterweighted gate at the bottom which opened and discharged its load when the cone was nearly filled with sand. This received the sand and water from the sand screen; the sand settled to the bottom, and the surplus water overflowed a notch in the lip and ran away. For some reason this device never worked satisfactorily at any of the plants, as too much water went through it with the sand into the bins, and there was a marked increase in the water content of the sand in the bins whenever the washing plant was running. This was taken care of at Germantown by installing two tipples underneath this sand cone, so arranged that the first, after receiving two discharges from the cone, would dump automatically into the second, which in turn, after taking two discharges from the first, would dump into the bin. Each of these tipples was so arranged that surplus water would drain out of the sand and pass off while this operation was going on. This apparatus was simple, easy to install, required very little attention and worked satisfactorily. At Englewood, and at some of the other dams, the trouble was corrected by catching the discharge from the sand cone in an inclined trough, in which a short piece of screw conveyor pushed the sand up the incline and thence into the bin, while the surplus water drained out through holes in the bottom of the trough and was carried away. Some of the finest of the sand was lost by these operations, which ordinarily made no difference, but in two cases (Huffman and Lockington), where there was a scarcity of fines in the sand to begin with, it affected the working of the sand in the mix and had to be compensated for, as will be explained later.

Mixing. Adjustable measuring boxes, built into the chutes between the bins and the charging hopper above the mixer, permitted such changes in mix as were desired, but generally speaking the capacities of the boxes were seldom changed, and the different grades of concrete were obtained by varying the amount of cement per batch. The capacity of the sand box at Germantown was fixed at 12½ cu. ft.; the fine and coarse gravel boxes were adjusted to give a combined output of 25 cu. ft. for the conduit linings and arches, and 28 cu. ft. for the leaner mixes, although the ratio of fine to coarse gravel was changed from time

to time to allow for variations in run of the pit. It was found to be more economical to use up all of both sizes of gravel that came to the bins, rather than to hold to an ideal combination of fine and coarse gravel, which would have resulted in wasting the surplus of one or the other size from time to time. Furthermore, it will be remembered that it was possible to run the oversize material into the coarse gravel bin when desired, and it was the practice at Germantown to do this for all concrete except that in conduit linings and in thin arch sections, thus cutting down the waste from the gravel plant to a minimum.

With the sand content of a batch fixed at 12½ cu. ft. and the gravel at 28 cu. ft., a 5-sack batch gave a 1:2.66:5.96 mix, figuring the cement at 0.94 cu. ft. per sack (100 lbs. per cu. ft.). A 6-sack batch gave a mix of 1:2.22:4.97; a 7-sack batch, 1:1.90:4.26; and with the gravel content changed to 25 cu. ft., an 8-sack batch gave a mix of 1:1.66:3.32. The latter mix was used only in the conduit linings and the thin arch sections where an especially high grade of concrete was desired. The 5-sack mix was used in the backs of heavy retaining walls or in the bottoms of thick floors, or in foundations, where there was no exposure to the weather. The 6-sack mix was used in faces of walls or floors exposed to the weather, but not to severe frost action, or high velocities of flow, or destructive action such as occurs in the stilling pool. In the latter cases the 7-sack mix was specified. At Englewood the sand content of a batch was fixed at 13 cu. ft., and the combined volume of fine and coarse gravel was about 26 cu. ft. A 5-sack batch there gave a 1:2.8:5.6 mix, and an 8-sack batch ran 1:1.75:3.5.

Cement was brought to the mixer platform from storage, on warehouse trucks, and the sacks were opened and poured in to the charging hopper by a man stationed there for that purpose, who shook out the sacks and stacked up the empties for return to the cement house, where they were sorted and baled for return to the mills. It was the practice to have a card, hung up where it could be seen by the cement man and also by the inspector on the job, indicating the number of sacks per batch being run at the time. The mix was changed when desired, by order of the inspector, and this card was a check to insure against misunderstanding or negligence.

On account of the variation in the wetness of the materials in the bins, the water could not be accurately measured into the

batch. This put an additional responsibility on the mixer-man, but after a little careful training he was able to regulate the water content of each batch with close uniformity. Concrete was mixed to a "mushy" consistency leaning towards the dry rather than to the wet. The mixer was a Smith one-yard machine, electrically driven, geared to 10½ revolutions per minute. The practice was to hold each batch in the mixer for about 15 revolutions.

Forms. At Germantown the wall forms for the inlet and outlet structures were built up in place, but at Englewood they were made up in panels about 30 feet long and 5 feet high, with the studding held at the bottom by bolts in the concrete, and extended to act as cantilevers. Thus the lower panels could be released and used again as the walls were built up. The forms for the conduits themselves required considerable study in order that they might meet all the requirements as to accuracy, smooth true surfaces, strength, rigidity and reasonable progress of work, without involving too great expense. Because of this it is believed to be worth while to give a detailed description of them here.

The conduits were built in sections about 30 feet long, the interior dimensions of all sections being the same except at the upstream and downstream ends. Concreting was carried on in each section in five successive stages as indicated in Fig. 82, joints being introduced at each of the stages. The four joints thus introduced into the arches were for the purpose of avoiding high temperature stresses which would have occurred if fixed arches had been used. These joints made the construction of the arches much slower and more expensive than that of the walls, but this additional cost was believed worth while considering the added safety gained.

The conduits being built in a trench cut in rock, exterior forms were not necessary except for the arches, the rock itself taking the place of exterior forms for the walls and floor.

To save expense the twin conduits were carried forward in the construction simultaneously, with forms built up in sections 30 feet in length, the interior forms for each conduit being exactly similar, set side by side, and enclosed at the ends by transverse bulkheads. When the concrete of a section had set, the forms were slackened to detach them from the concrete, rolled forward 30 feet, and set up again. The forms rolled on truck wheels

attached to the bottom pieces, track being furnished by the already finished floor or, in the case of the upper forms, by shoulders in the lower walls. These shoulders also furnished side support to the floor of the permanent conduits.

The lower forms were built in two entirely separate halves, in order to permit them to collapse free from the finished concrete and roll forward in the manner just indicated. One of the halves is shown in the lower part of the left-hand portion of Fig. 83, and

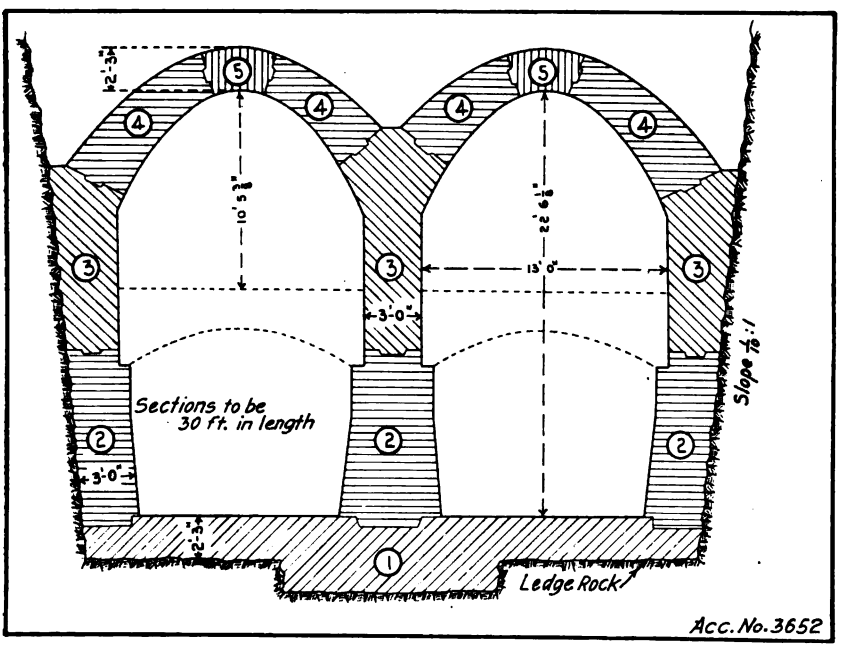


FIG. 82—STAGES OF POURING SECTIONS OF OUTLET CONDUITS
In each section concreting was carried on in five successive stages, joints being introduced at each of the stages. The four arch joints were for the purpose of avoiding high temperature stresses.

the two together, side by side, in the right-hand conduit in the same figure. Each of these independent sections consists of a set of cross frames held in position by longitudinal and diagonal bracing in both horizontal and vertical planes, thus creating vertical and horizontal trusses. The horizontal trusses are required to hold the form rigid against the pressure of the poured concrete, since each 30-foot section could be held rigidly in position only at

the ends. The two sections were held together at the center by steel plates on each side of the upper and lower chord members of the cross frames, and also by two bolts EE, through the inside vertical members of the cross frames. These bolts were threaded their entire length and had four nuts and washers each. They were used to draw the sections together when desiring to move ahead and also to force them apart when setting up in the new position. The steel plates on the chords had slotted holes so that when the nuts were loosened on bolts M the sections could be drawn together. When the forms were in position for concreting, reliance was placed neither on the bolts M through the plates nor on the adjusting bolts E, but upon oak wedges, driven between the ends of upper and lower chords along the center line. (The wedges correspond to those marked "W" in the upper form.) When the form was to be moved ahead the bolts M were loosened, the wedges driven out, and the two sections drawn together a sufficient distance by means of bolts E, then raised by a set of jacks, and a set of heavy truck wheels placed in position as shown in the left-hand conduit. The sections were then lowered until the truck wheels rested on the floor, and the form was moved ahead to the new position and set by reversing the processes.

The lagging consisted of 2-inch dressed pine and was not protected by steel sheathing. The forms were in fact built of wood throughout, steel being discarded as too expensive. With a little care in the selection and handling of the lagging a perfectly true and smooth surface was obtained. At Englewood these lower forms were set up and used 22 times in succession; at Germantown, 18 times; and except for the lagging becoming somewhat rough by the spreading of the concrete along its surface, they were still in good condition when the conduits were finished.

The weight of one complete 30-foot lower section for one conduit was 10 tons.

The upper interior forms consisted of two main parts, a rectangular part for the upper side walls and a segmental part for the arches. The general design of the rectangular portion was similar to that of the lower forms, including the two separate halves with the steel side plates, wedges and adjusting bolts E. There being no supports under the center, however, the side plates on the lower chord members were made 8 feet long, in order to keep

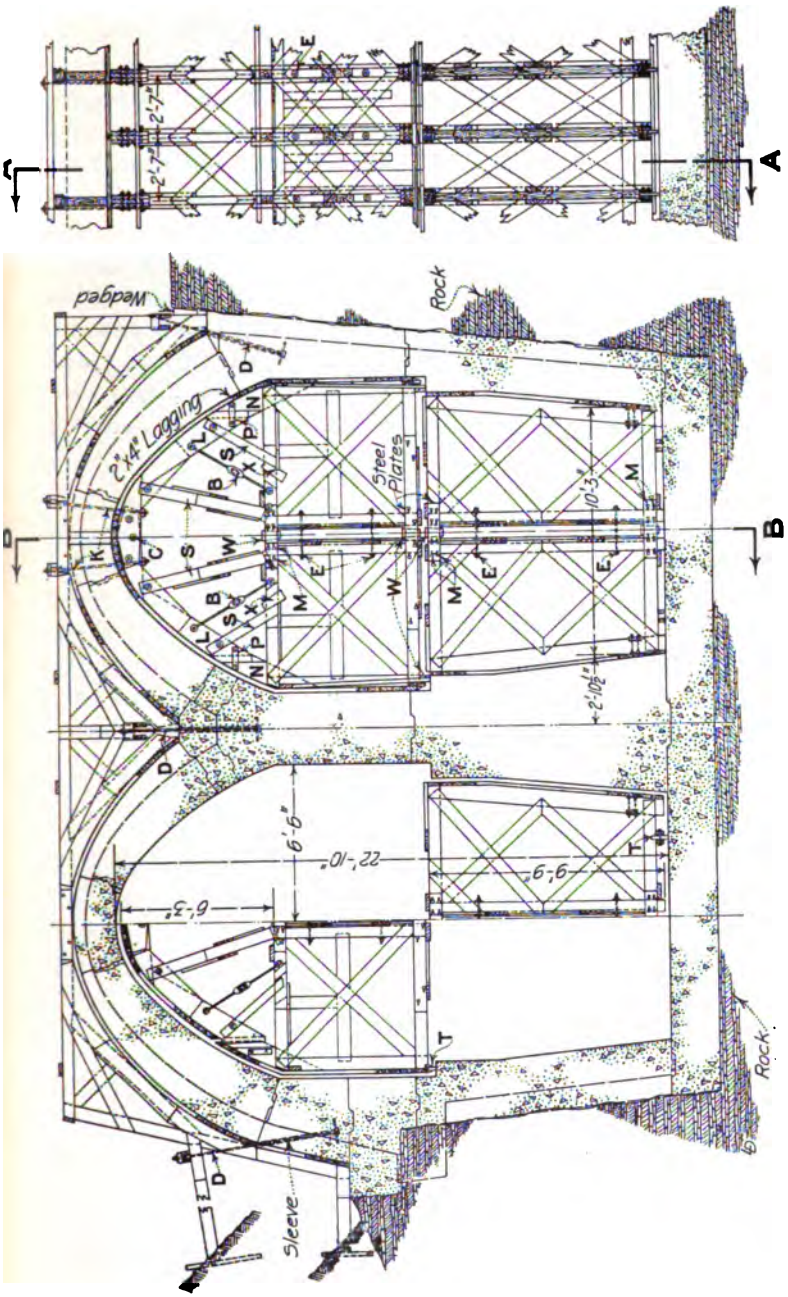


FIG. 83—FORMS FOR OUTLET CONDUITS

The lower forms were built in two separate similar halves. One of these halves is shown in the lower part of the left-hand portion of the figure, and the two together in the right-hand conduit. The upper interior forms consisted of two main parts, a rectangular part for the upper side walls and a segmental part for the arches. The exterior forms were made in three parts, one for each of the two inner halves of the arches and one for each of the two outer halves.

this member in line. The truck casters on which the form rolled forward are indicated at T, to the left in Fig. 83. To facilitate collapsing, the arch ribs of the segmental part of the form were built in four parts. The triangular pieces NN at the lower ends were rigidly attached to the rectangular form below. The segments LL were pivoted together at C. They were bolted together also by the bolts adjacent to C. LL and NN were connected by slotted straps PP tightened by bolts and nuts. The weight of these parts was carried to the rectangular form below by means of the diagonal struts SSSS, the lower ends of these bearing directly upon the upper chord of the form, and the pieces LL were held rigidly in place for the concrete by forcing the lower ends of the struts away from the center line of the forms by means of threaded straps for the two inner struts, and by wedges for the two outer ones. (See Fig. 83.)

To collapse the forms, the straps PP are loosened, the straps and wedges holding the bottoms of all four struts are loosened, and the strut bottoms driven toward the center line of the forms. The turnbuckles BB are then tightened, pulling down and in on the two pieces LL, the latter turning on the pivot C, the other two pins near C having first been pulled out. The rectangular part of the form is then collapsed like the lower form, drawing the entire upper form away from the walls and down from the arch, permitting it to be rolled forward on its casters into its new position. Carrying the weight of the arch form by the rectangular section below necessitated making the latter into a very rigid truss. This was done by means of the diagonal braces shown. The arch ribs consisted of three layers of 2-inch plank cut to shape and spiked together, the layers interlocking where the parts of the rib were pivoted together. The interior and exterior arch forms were bolted together by the bolts KK. These bolts were removed, of course, to collapse the interior form.

One 30-foot section of upper interior forms weighed 15 tons, yet in many cases the moving ahead took only about eight minutes.

Both the upper and lower forms were made sufficiently long for one end to bear inside on the finished concrete of the preceding section and for brackets to be bolted upon the other end to hold the end bulkhead. These brackets were detached and the bulkhead removed before collapsing the interior forms.

In rolling the forms forward during construction, many of the truck wheels (used on the lower forms) and casters (used on the upper forms) were broken. This was due to the irregularity in the elevation of the concrete upon which they rolled. This would sometimes throw a load many times as great as was intended on the wheels or casters and break them. A special wheel and caster were then designed and no further trouble was experienced.

It will be noticed by referring to Fig. 82 that when the concrete was poured in the lower portion it was carried about 8 inches above the side shoulders to act as a guide in setting the upper forms. This was very easy to do and was very important.

The number of forms to be provided was based on the estimate that one floor section could be placed every 2 days, one lower side wall section every 3 days, and one upper side wall and arch section every 6 days. One set of lower interior and two of upper interior forms were therefore provided. One set of upper forms was planned to be set up over the first completed section of the lower portion as soon as the latter had aged 2 weeks. A second upper set of forms was planned to start when the lower forms, which began concreting at one end of the conduits, had passed the center line of the dam. In this way the upper and lower parts of the conduits would be finished about the same time.

The general design of the exterior forms is indicated in Fig. 83. The conduits being built in deep rock excavation, exterior forms were necessary only for the arches. To facilitate handling, they were designed in three parts, one for the two inner halves of the arches and one for each of the two outer halves. The parts were held together at the conduit crowns by steel side plates in the same manner as the interior forms, except that the holes in the plates were not slotted. They were held down by bolts D along the center line of the twin conduits and along the outside of each conduit, and were rigidly held to the interior arch forms by means of the bolts K at the crown. The holding down bolts D passed through large plate washers buried in the concrete at the lower end, with a nut on the under side of the plate, and were incased in a tin sleeve in the concrete so that they might be unscrewed and taken out when there is no further use for them. The concrete surrounding these plate washers, as a rule, was only 36 to 48 hours old when the haunch arch sections were poured. On this account these plate washers had to be very large so that they would hold almost solely by the weight of the concrete above

them. After the haunch sections had set for a day the bolts K were taken out and the crown key section was concreted without a top form.

After the upper interior form was moved ahead, the exterior form was taken apart and moved ahead in sections, then reassembled in position over the interior form and the bulkhead forms set in position. On account of the variation in thickness of the arches, this thickness increasing steadily from the ends of the conduits to the center line of the dam (to give support to the

FIG. 84—COMPLETED CONDUITS AT GERMANTOWN
This view is looking downstream, with the concrete track at the left.

steadily increasing weight of the superincumbent earth in the finished dam), the same exterior form could be used only a few times before being rebuilt.

Placing Concrete. The mixer discharged into 1¼-yd. side hopper dump cars, which were hauled in trains of two or three cars each by 3-ton gasoline locomotives on a 36-in. gage track built along the side of the cut parallel to the conduit location. At Germantown the track was located high enough so that practically all of the concrete could be put into place by means of

chutes. For the small amount that could not be placed by chutes, concrete buggies were used. Fig. 84 shows the completed conduit (looking downstream) with the concrete track at the left. At Englewood there was not as much elevation to be had for the concrete track, so that more of the concrete there was placed by other methods as will be explained later.

The concreting started at the downstream end of the stilling pool and followed the excavation as it proceeded upstream. The

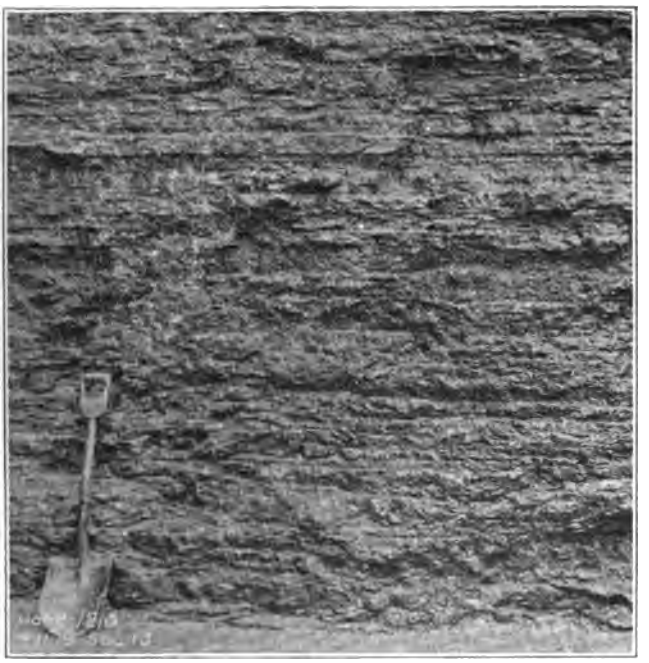


FIG. 85—ROCK TRENCH PREPARED FOR CONCRETING
Where concrete was placed against the rock walls they were cleaned and brushed so as to remove any loose material or dirt.

floor was laid in alternate panels, with contraction joints not more than 30 feet apart either way. The walls were also built with contraction joints about 30 feet apart, and these joints in the walls were so spaced that they connected with similar joints in the floor.

Just prior to the placing of concrete in any section the bottom was thoroughly cleaned and brushed, so as to take up any loose

material or dirt that might prevent a good bond with the foundation. A wash of neat cement grout was then brushed in, and the first few batches of concrete were given an excess of mortar to insure a good bed of rich concrete, or in some cases the proportion of coarse gravel was cut down in the first few batches. The sides of the rock walls, where concrete was to be placed against them, were cleaned with the same care as was the bottom, and where a layer of fresh concrete was to be placed on concrete already set, the same care was used to remove laitance and any dirt that may have accumulated. Fig. 85 shows one of the rock walls of the conduit trench at Englewood cleaned, ready for the placing of concrete.

Generally the concrete was dumped from the cars into chutes, through which it passed directly into the forms. For most of the work the chutes were set at an angle of about 20° to 25° with the horizontal, at which slope the concrete flowed down into place by gravity, and smoothly enough to avoid separation of materials. A steeper slope tends to cause a separation of the gravel from the mortar; mixing the concrete too wet has the same effect, to say nothing of the harm to the quality of the concrete itself. For some of the long reaches, where the slope was flattended below about 20°, the concrete had to be helped along with shovels. When this became too laborious, concrete buggies were used, and for the walls of the inlet structure, both at Germantown and Englewood, a tower hoist and chute were utilized. The author's experience in chuting concrete, is that best results, both as to character of concrete and ease of operation, are obtained by using a concrete of "mushy" consistency and holding the chute at an angle of about 22° (2½ to 1 slope). A wetter mix causes trouble from separation of materials, irregular flow, clogging and overflowing, fully as annoying as the troubles caused by a mix that is too dry. The mix that works best in the chutes is the same "mushy" mix that works up best in the forms.

The chutes were supported by a traveling truss which spanned the conduits, traveling on a rail on either side. This truss was used also for moving and setting the outside conduit forms. At Englewood, where it was not possible to get as much elevation for the concrete track as at Germantown, a raised platform was built on the near side of these traveling trusses, and for reaching the work on the far side of the conduits the chutes were raised to take the concrete from this platform, and the concrete cars

were hoisted to dumping position by a locomotive crane which was available for that service in connection with other uses. Fig. 86 shows the trusses and the partly completed work at Englewood. To properly reach the far walls of the stilling pool at Englewood, a guy derrick was set up and used not only for distributing concrete but also for handling the panel forms.

FIG. 86—TRAVELING TRUSSES FOR SUPPORTING CONCRETE CHUTES AT ENGLEWOOD

These trusses spanned the conduits, traveling on a rail on either side. They were used for moving and setting the outside conduit forms as well as for supporting the chutes. The guy derrick in the background was used to reach the far walls of the stilling pool.

The concrete was well tramped in the forms by the men who did the spading. The spading and tramping was done with great care, especially along the exposed faces of work which were to be subjected to high velocities of flow, or severe frost action. Nothing short of perfect work was acceptable in those places and every reasonable effort was made to secure it. And these efforts were so successful that practically no patching was required in

the conduits when the forms were removed. As an added precaution, however, the conduit linings were gone over with carborundum bricks, and washed with neat cement grout which was again rubbed in with the brick, so as to secure an absolutely smooth and dense surface to resist the erosive action of the water discharging through these conduits. The velocity of the water at times would be as high as 50 feet per second.

In pouring the heavy walls, where the body of the wall was made of a leaner mix than the exposed face, the procedure was as follows: A layer of the lean mix was started at the back of the wall, for the full length of the section (usually 30 feet) and was brought forward to within about 2 feet of the face, when the richer mix was called for. The mixer man acknowledged the call by changing the indicator card displayed at the mixing plant, and that mix was used to complete the layer along the forms and to start the next layer back, when the leaner mix was again called for. In this way, a sharp line of demarkation between the two grades of concrete was avoided.

Placing Permanent Floor. The placing of the permanent floors of these conduits (see Fig. 80) was an entirely separate operation, as it was only after the dam was nearly finished that the conduits could be cut down to their final size. At Englewood the same plant was still available as was used for the other work, but at Germantown the original plant had already been disposed of, so a special arrangement had to be made for this work. Sufficient sand and gravel had been prepared and set aside in storage piles before the screening plant was dismantled, and a temporary setting of a 1-yd. mixer was made in a convenient position on one of the walls of the inlet structure.

The first step was to divert the low water flow through one conduit by means of simple cofferdams provided for in the original design. (Such diversions may be desirable at intervals in the future, for inspection purposes.) Then the one conduit was pumped out, and the concrete bulkheads, across the lower portion at both ends, were poured. These bulkheads were designed to reinforce the permanent floor, and to prevent any possible flow or leakage below it which might in time dislodge some of the supporting material. One of them is shown just under the headwall in Fig. 81. The one at the upstream end was thicker than this one, as the wear and tear on the floor would be more severe there. A part of the purpose of this also was to prevent the

transmission of hydrostatic pressure from the reservoir to below the conduit floor.

Next, the lower parts of the conduits, up to floor level, were filled with sand and gravel taken from the main borrow pits and pumped into the conduits from the hog-boxes. This made a very compact fill on which the concrete floor could be laid at once. The floor was so designed, however, that it required no permanent support from the material below it (see Fig. 80).

For the distribution of the concrete, a narrow gage track was laid through the conduit, and the mixed concrete was hauled in 1½-yd. side dump cars by the 3-ton gasoline locomotives. The track was supported on cross timbers posted up from the offsets in the side walls. As the concrete was placed the posts were removed, and after the concrete had hardened the cross timbers were blocked up from the finished sections of floor. The floor was laid in alternate sections, 30 feet in length, the contraction joints being located so as to connect with those in the walls. The base of this floor was made of the 6-sack mix, while the wearing surface, about 12-in. in thickness, was made of the same mix as the permanent conduit linings, i. e., 1:1.66:3.32. No forms were required, except the simple bulkheads at the ends of the sections. The floor was given a good trowel finish. As soon as the last of this concrete had had about a week to set, the water was turned through it (small flow and low velocity), and the same procedure was followed in the other conduit.

Costs. The costs of this work at Germantown and at Englewood are given in the following tabulations. During this period the rate for common labor was from 38c to 42c per hour. Carpenters were paid 60c to 75c per hour, carpenter helpers 40c to 44c per hour, all men on an hourly basis being given 11 hours' pay for 10 hours' work (time and a half after 8 hours), which virtually increased the foregoing rates by 10%. The average cost of cement was $2.27 per barrel including freight and loss on sacks. The amount of cement used per cubic yard of concrete at both jobs was about 1.6 barrels. Form lumber was worth about $35 to $40 per M ft. B. M.

The cost of superintendence at Germantown was higher because a large percentage of the superintendent's time was charged to this work as there were fewer operations going on at the time than was the case at Englewood. The higher labor cost at Germantown is accounted for by the fact that there was no

railroad connection in to the work and all of the materials and supplies had to be hauled over hilly roads for a distance of about 3½ miles. Most of this hauling was a charge against the labor account. Taking these things into consideration the unit costs of these two jobs are remarkably close.

Cost of Concrete Outlet Conduits at Germantown Dam
(16,350 cu. yds.)

		Cost per Cu. Yd.	Percent. of Total
Superintendence and Overhead	$ 15,359.64	$ 0.94	5.0
Labor	147,334.20	9.01	43.1
Cement at $2.27 per barrel	59,383.20	3.64	19.4
Supplies (including form lumber, coal, electric power, etc.)	69,750.03	4.27	22.8
Plant Depreciation (including installation and dismantling)	14,617.80	.89	4.7
	$306,444.87	$18.75 per cu. yd.	

Cost of Concrete Outlet Conduits at Englewood Dam
(17,850 cu. yds.)

		Cost per Cu. Yd.	Percent. of Total
Superintendence and Overhead	$ 13,157.75	$ 0.74	4.2
Labor	135,316.25	7.58	43.1
Cement at $2.27 per barrel	64,831.20	3.63	20.7
Supplies (including form lumber, coal, electric power, etc.)	83,187.86	4.66	26.5
Plant Depreciation (including installation and dismantling)	17,284.46	.97	5.5
	$313,777.52	$17.58 per cu. yd.	

Outlet and Spillway Structures at Taylorsville, Huffman and Lockington Dams

Description: The type of structure used at these dams is very different from that just described, and required quite different methods of construction. A reference to Fig. 27 (Chapter III) will bring to mind again the essential features of this type, namely, a gap through the dam formed by two massive retaining walls, facing each other, with a floor between; a spillway weir across the gap, with outlet conduits piercing the lower part of the weir, at river level. The coping profile of the retaining walls fits the cross section of the dam, in each case, and

there is a stilling pool, downstream from the conduits, similar to those at Englewood and Germantown. The maximum height of the retaining walls at Taylorsville, is nearly 90 feet, requiring a base thickness of about 50 feet; the width of floor at the throat of the structure is 95 feet, and the width along the spillway crest is 132 feet. The conduits are four in number, each opening measuring 15 feet across, and 19 feet high. The greatest width across the stilling pool is 230 feet and the total length of the concrete structure is 555 feet. The design provided for gravity walls throughout. The total amount of concrete, exclusive of the bridge, was nearly 48,000 cu. yds. and this was the largest single concrete structure built by the District.

The Huffman structure has three conduit openings, each 15 feet wide and 16½ feet high. The maximum height of the retaining walls there is about 80 feet. The width of the floor at the throat of the structure is 67 feet, and along the spillway crest the width is 100 feet. The greatest width across the stilling pool is 195 feet. The total length of the structure is 540 feet.

The Lockington structure is considerably smaller, being only 440 feet long, and having a maximum width, across the widest part of the stilling pool, of 120 feet. There are two 9-ft.x9-ft. conduit openings, and the throat of the structure measures 28 feet wide. At the crest of the spillway the width is 72 feet. The maximum height of the retaining walls is about 70 feet.

Reference is made again to Parts III and VII of the Technical Reports, where discussions are given of the main features of design of the outlet structures, and of the theory of the hydraulic jump.

Washing and Screening: The washing, screening and mixing plants at these three dams were duplicates of the ones used at Englewood and Germantown, a description of which has already been given in Chapter III and illustrated in Fig. 25. Special attention was given to the screening operation at Taylorsville because of the fact that there was an objectionable surplus of pea gravel and buckshot sand in the concrete stock, as obtained from the borrow pit, which gave a harsh, gritty quality to the concrete making it difficult to handle in the forms. It required some study in order to obtain a mix which was workable and at the same time economical. This was done by removing the excess of these sizes from the screened material. By diminishing

the force of the jet of wash water at the lower end of the sand screen, the coarser of the sand was carried over the screen to the fine gravel bin. A coarse sand screen was installed there which took out the sand and most of pea gravel before the gravel was discharged into the bin. This sand and pea gravel was then wasted. Inasmuch as the pit yielded surplus sand which had to be wasted anyway, this method did not involve much expense, and a well graded aggregate was obtained.

At Huffman, the somewhat unusual condition was encountered of a shortage of sand in the borrow pit material, and as is often the case under such condition, the sand was deficient in fines. Wasting a percentage of the coarser sands in order to cut down the average fineness was not practical here because of the shortage of sand in the pit. About all that could be done, then, was to take steps to save all of the finer sand, some of which might otherwise escape with the wash water, and this was done by means of settling boxes in the waste water lines. Even so, the gradation of the sand was not fully satisfactory, and more cement had to be used, in order to obtain a workable concrete, than otherwise would have been required in the leaner mixes. Moreover, the shortage of sand persisted to the extent that gravel had to be continually wasted at Huffman. Not enough sand was available so that all of the gravel could be used up. To further improve the situation the addition of hydrated lime to the leaner mixes was tried, and this resulted in a considerable saving of cement. It was found that this hydrated lime could be substituted for the excess cement otherwise required, with as good results as far as workability of the concrete was concerned, and with satisfactory results as to quality. The hydrated lime was needed only in the leaner mixes, because in the richer mix such as was used on exposed faces, (about 1:2:4) there was cement enough to give a good workable concrete. In spite of the additional cost of introducing another ingredient to the mix, the use of hydrated lime in this case was beneficial in making up for the shortage of fines in the sand.

At Lockington there was a surplus of sand in the pit, but in the sand itself was a preponderance of the coarser particles, and the grading was much improved by removing and wasting a portion of the coarse sand. Inasmuch as there was a surplus anyway, this simply meant the additional work of removing the coarser particles instead of wasting the sand as it came. This

was accomplished by a simple arrangement of screens in the washing plant. It was found desirable also to trap all the fine sand which otherwise would be carried away with the wash water, as was done at Huffman, so that all the fines would be retained, as far as possible, and the gradation thus improved.

The trouble of having a lot of water in the sand while the washing plant was in operation, and much less at other times, which was common to all of the screening plants until corrected, was overcome at Taylorsville by the novel method of adding *more* water, at the measuring boxes, until the sand became saturated whether the washing plant was running or not. While the men at the other jobs preferred the methods, already described, of removing most of the water, still the force at Taylorsville insisted that their own plan was simple, and just as good. At any rate, a first class grade of concrete was turned out, and that was the object sought. The relative quantity of sand in the borrow pit had its effect on this surplus water problem also, because at Huffman, where there was a deficiency of sand in the pit and therefore the sand bin was almost always drawn down, the sand usually passed almost directly from screens to mixer, with little chance to drain out in the bins; in that case, before the screw conveyor was introduced, so much water came through with the sand that the concrete would be too wet even if no more water at all were added at the mixer. At Taylorsville, where surplus sand was continually being wasted, the sand bins usually were full or nearly so, and the water had a better chance to drain out, therefore the sand was never so wet that it carried enough water for the whole mix.

Mixing: The mixing of the concrete was handled at Taylorsville, Huffman and Lockington in the same manner as at Germantown and Englewood, changing from one mix to another, as required, by changing the number of sacks of cement per batch. However, at these three dams, where the heavy retaining walls (some of them 50 feet thick at the base) comprised such a large proportion of the concrete work, much more of the leaner mixes were used. The principal consideration there was weight and stability, rather than strength or density, except in the exposed faces where a richer mix was used as explained heretofore. In many places a mix as lean as 1: 3¼: 6½ was suitable. The concrete in exposed faces was mixed in proportions approximating 1:2:4, and the conduit linings and wearing

surfaces of floors were made about 1:1½:3. This rule was followed in most of the heavy sections, that with the mortar properly proportioned for the work in hand, as much gravel was added as the mortar would take and still produce a workable mix. In other words, the mortar would take more gravel, theoretically, than is practical from the standpoint of workability in the forms.

FIG. 87—LAYOUT OF CONCRETE DISTRIBUTING PLANT AT TAYLORSVILLE

The material was hauled from the gravel pit across the trestle on the right to the screening plant. Trackways leading from the screening and mixing plant to the job may be seen in the middle distance, and the two movable, electric, stiffleg derricks for handling the mixed concrete and form panels are in the foreground. The work started at the downstream end of the structure.

Distributing Plant: In Chapter III is a description of the concrete distributing plant at each of the dams. Fig. 87 shows the layout for the Taylorsville work. The steam dragline at the gravel pit is visible, in the distance at the right. The material was hauled from there across the trestle, to the screening plant, in 12-yd. dump cars, by standard gage dinkeys. The trackways leading from the screening and mixing plant to the job may be seen in the middle distance and the two movable,

electric, stiffleg derricks for handling the mixed concrete and the form panels, are in the foreground of this view. Bottom dump buckets on trucks, hauled by small gasoline locomotives brought the concrete from the mixer to the derricks. The work started at the downstream end of the structure and at the time this picture was taken the work was about half finished. The massiveness of the walls is seen by a glance at the right hand wall in this view. The location of contraction joints is shown

FIG. 88—LAYOUT OF CONCRETE DISTRIBUTING PLANT AT HUFFMAN

Here the work was started at the upstream end. The view is looking downstream from the screening and mixing plant.

also. Here, as at the other dams, these joints were established about 30 feet apart in all walls, floors, and other units of the structure.

At Huffman the layout was similar, except that the work started at the upstream end and therefore the screening and mixing plant was located at the upper end of the work. Fig. 88 shows the Huffman layout, with the concrete distributing tracks following along the foot of the walls on either side, and the der-

ricks in the center. Here the two derricks were mounted on a single base, and therefore moved down the line simultaneously. This was practicable at Huffman as there was less difference in yardage of the two walls than at Taylorsville. The illustration also shows the dragline machine, in the distance, completing the excavation at the downstream end of the structure. The

FIG. 89—LAYOUT OF CONCRETE DISTRIBUTING PLANT AT LOCKINGTON

Two guy derricks, each with 105-ft. boom, were able to cover the whole structure. The work is proceeding downstream, and the view is from the screening and mixing plant.

screening and mixing plant is at the location from which the picture was taken.

The Lockington work is illustrated in Fig. 89. Here, two guy derricks, each with a 105-ft. boom, were able to cover the whole structure, one of them being used also for a part of the rock excavation from the stilling pool. This work is proceeding downstream, as at Huffman. The screening and mixing plant is just out of the picture in the foreground. The raw gravel from the excavation, stored in the pile at the left, will be used later when

the large dragline machine gets out of reach. The dragline may be seen in this view, completing the excavation for the outlet channel; at the same time it loads the four-yard cars, on the track at the left, which are brought to the screening plant by the narrow gage dinkey locomotive. The concrete distributing tracks follow along the foot of the wall, as at Huffman.

Forms: Cantilever forms made up in standard panels were used for practically all of the wall construction.. Fig. 90 shows the essential details of these panel forms as used at Taylorsville and at Huffman. The principal differences are: (1) size of panels, (2) size of anchor washers, (3) method of handling bolts. The first and last of these were due largely to personal preferences. As to the size of anchor washers, the 6-in. washer is more conservative but considerably more expensive, and during the time this work was going on, these washers cost nearly 50c apiece. By careful handling the 4-in. size was used successfully at Taylorsville, but very likely that size is close to the limit of safety. At Taylorsville where the four different sizes of bolts were used, it gradually became the practice to discontinue the use of the "D" bolts and use instead more "C" bolts with sleeve nuts. As these bolts at Taylorsville were not protected from the concrete by tubes or wrapping, all those that were to be removed from the concrete were coated with an oil made up as follows: One part petroleum jelly, one part black transmission oil, two parts coal oil. It was necessary to give particular attention to these bolts as they had to be given a turn within a few hours after the concrete was placed in order that they could be removed later. It finally became the practice to make one man personally responsible for this so that none of the bolts would be overlooked. At the other jobs where the tin tubes were used, this preliminary turning of bolts was not necessary.

This oil that has just been mentioned, was used also to oil all forms before the fresh concrete was placed against them. Each panel was thoroughly cleaned and given a fresh coat of oil every time it was used. At Taylorsville, 20 sections of the standard panel forms were in use, and all of them were in good condition when the job was finished.

At Lockington the same general construction and methods of handling forms were used as were in use at Huffman (see Fig. 90, lower view), but the standard panel at Lockington was 10

ft. high and 27 ft. long. This length fitted the spacing of the contraction joints there, and although the panels were 10 ft. high it was never the practice to pour more than a 5-ft. lift at one time. With the guy derricks always available to handle these forms, the large panels were quite satisfactory although at the other jobs it was preferred to have them in smaller units.

At some places where the side wall was nothing more than a facing 2 or 3 ft. thick against the rock, special care had to be taken so that the forms would not spring, as the whole pouring might be made within an hour or two, before the concrete would have a chance to harden. At Taylorsville where the lighter studding was used, it was customary in such cases to use an extra 4 x 10 to double the middle studding. (The end studs were bolted to corresponding studs on adjoining panels, thus giving them additional support).

Special forms were required, of course, for the noses and ends of the piers between the conduit openings and for the conduit arches. The conduits at these three dams, where the combination outlet and spillway structures were used, were comparatively short and therefore the forms for these arches were built in place.

Placing Concrete: The same precautions that were mentioned in the description of the Germantown work were followed in the mixing and placing of the concrete at the other dams. Special care was taken to see that the concrete was turned out of proper consistency, and the methods which have already been mentioned for cleaning up the foundations and preparing old concrete surfaces for a continuation of the work, were followed out. Careful attention was given the spading, and if it happened that free water was present (an indication that the mix was too wet) special care was taken to see that no spading was done when any of this water was standing against the forms, as that would result in pumping out the cement and leaving a rough spot on the face. The facing of the heavy walls with a richer mix of concrete was accomplished in the same manner as has been described in connection with the work at Germantown and Englewood. Most of the concrete was placed in the forms by the derricks but at Huffman, where the derricks were mounted on the same base, it sometimes happened that the floor sections should be carried forward when the derricks might be held back to finish a high wall section. In such cases the floor sec-

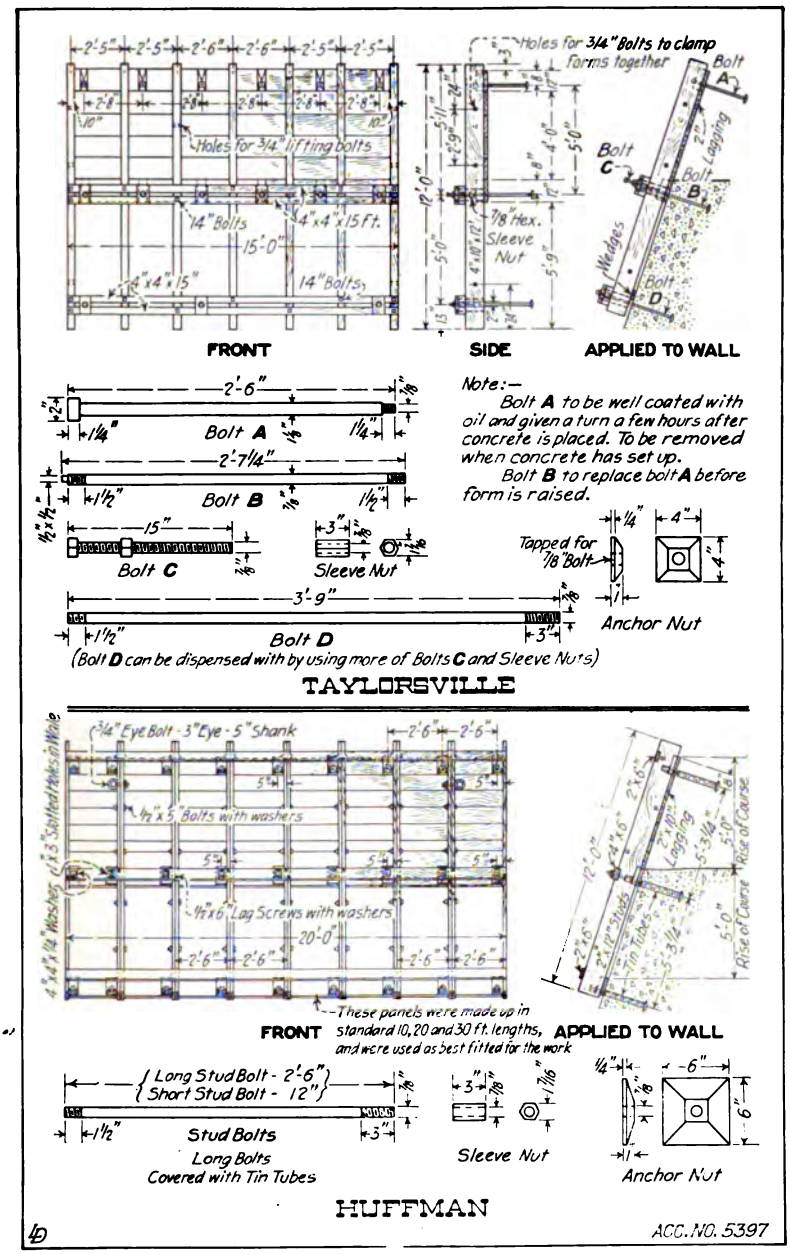


FIG. 90—DETAILS OF CANTILEVER PANEL FORMS USED AT TAYLORSVILLE AND HUFFMAN

tions were laid by means of an elevated hopper and chutes, set as far ahead as the boom would reach, the hopper being fed by one of the derricks with the regular dump buckets. This hopper and chutes could be moved about easily by the derricks.

Procedure at Taylorsville: The placing of the walls and floors and enough of the piers to bring them above the normal water level, was the first step in the construction of the Taylorsville structure. The gap thus formed was used for river control during the construction of the dam, and after the hydraulic fill had

FIG. 91—CONSTRUCTION OF CROSS DAM AND COMPLETION OF CONDUITS AT TAYLORSVILLE

A stiffleg derrick on the slope above the mixing plant picked up loaded buckets from the mixer and swung them into dumping position in the forms. The suspension bridge in the foreground carried a dredge pipe line across the river to the dam.

been carried to a sufficient height, (nearly to completion), the structure was completed by building the cross dam forming the spillway lip, and completing the conduit openings. Sand and gravel for this work was screened and laid aside at the time the first work was being done, as the original concrete plant was to be disposed of before this last work was undertaken. This work was done from a special mixing plant set up for the purpose on top of one of the side walls of the stilling pool so located that the sand and gravel could be hauled in from storage piles and fed to

it from the cars by gravity. A stiffleg derrick was erected on the slope above the mixing plant in such position that it could pick up the loaded buckets from the mixer and swing them into dumping position in the forms. Having the intermediate piers previously brought up above water level, it was possible to support most of the centering for the conduit arches from the sides of the piers, but a row of center posts was required, in each case, extending down to the floor. The work was so arranged that at least one bay was always open, through which heavy drift could be passed in case of flood. As soon as the concrete had been brought up 6 or 8 feet above the tops of the arches, the rest of the cross dam could be built with little danger of interference from high water. Fig. 91 shows this part of the work under construction.

Costs at Taylorsville: The total quantity of concrete in the Taylorsville structure was 47,350 cu. yds. and the average unit cost was $10.18 per cu. yd. This was the final cost after all adjustments were made, and is based on cost of all concrete placed (including about 4% overbreak), divided by the "pay" quantity only, that is, measured to neat lines only, as provided in the specifications. Following is an analysis of the costs with some further figures covering labor distribution, cost of gravel production, etc.

Unit Cost of Concrete—Taylorsville Dam

47,350 Cu. Yds. Net

		Cost Per Cu. Yd.	Percentage of Total
Superintendence and Overhead	$ 16,257.94	$0.34	3.4
Labor	213,280.70	4.50	44.2
Cement at $2.25 per barrel	117,201.15	2.48	24.4
Power (elec., coal, gasoline)	17,994.52	.38	3.7
Supplies	68,898.83	1.45	14.2
Plant Depreciation (including installation and dismantling)	48,793.48	1.03	10.1
	$482,426.62	$10.18	

Laborers were paid from 38c to 46c per hour, carpenters 70c to 80c, carpenter helpers 40c to 50c, all with 11 hours' pay for 10 hours' work (time and a half after 8 hours) which really increased these labor rates by 10%. Cement cost $2.25 per barrel including loss on sacks and the price of lumber averaged about $40 per M ft. B. M. The amount of cement used per cubic yard

of concrete was 1.1 barrels including what was used for washing and patching. A distribution of labor costs on the job over a long time period gave the following percentages which have been applied to this final labor cost of $4.50 per cubic yard.

Labor Distribution Per Cubic Yard of Concrete in Place

	Percentage of Total	Cost Per Cu. Yd. of Concrete
Track Work in Borrow Pit	3.5	$0.157
Excavating Gravel and Sand	9.3	.418
Screening and Washing Gravel	5.3	.239
Unloading Cement	1.3	.059
Mixing and Placing Concrete	15.2	.684
Building and Setting Forms	26.2	1.179
Track Work on Distributing Track	5.9	.266
Repairs	1.8	.081
Heating and Miscellaneous	14.0	.630
Industrial Insurance	2.5	.112
Dismantling and Cleaning Up	15.0	.675
	100.0	$4.50 per cu. yd.

The "Supplies" item of $1.45 per cubic yard included lumber, bolts and nails, repair parts, and all miscellaneous supplies.

The "Power" item, 38c per cu. yd., was made up as follows:

Coal @ $4.20 per ton (Dragline at borrow pit, dinkeys, heating)	$0.13
Electric power @ 0.02 per k.w.-hr. (washing, screening, mixing, derricks, lighting)	0.23
Gasoline and oil	0.02
	$0.38

The form work averaged 4.7 sq. ft. of form surface per cu. yd. of concrete. This varied, on different parts of the work from 2.9 sq. ft. per cu. yd. in floors, (the stepped incline to the stilling pool, and bulkheads) to 15.5 sq. ft. per cu. yd. for the center wall of the stilling pool. For the heavy west wall of the structure it was 5.2 sq. ft. per cu. yd. Labor cost on forms, (building, setting up, and removing), averaged 20.6c per sq. ft. of surface covered.

The unit cost of excavating, hauling, washing and screening the gravel and sand for concrete was as follows:

Superintendence and Overhead	$0.10
Labor	0.44
Supplies	0.06

Power (Coal, elec. power, etc)	0.10
Plant depreciation	0.40
	$1.10 per cu. yd. (borrow pit measurement)

A good average month's work at Taylorsville was about 6,000 cu. yds. of concrete. The best month's record was 7,927 cu. yds. and the next best record was 7,025 cu. yds. During the winter months, and at the beginning and end of the job, the monthly output was considerably less than the average.

Procedure at Huffman: It was not practicable, at Huffman, to build the bottoms of the piers between the outlet conduits at the time the floor and walls were built, as was done at Taylorsville. All of the waterway was required for the handling of floods during construction. Therefore, when it came time to complete the structure, these piers had to be started at floor level. This required a carefully constructed cofferdam, as normal low water stood about 3½ feet deep on the floor. The details of design for this pier cofferdam are shown in Fig. 92. The bottom dimensions of each of the two piers were 11 ft. wide by 100 ft. long. The overall dimensions of the cofferdam were 24 ft. by 117 ft. The frame work was erected on the bank of the river, launched and floated into place, where the sheeting was set and the space filled with clay. This cofferdam was absolutely water tight, and after it had once been unwatered no more pumping was required. After the first pier had been completed to a safe elevation above the water line, the whole cofferdam was dismantled, taken out, reassembled, and floated into position for the other pier. The cost of the first installation of this cofferdam was $2730, including all labor and materials and a proper share of overhead and general expenses. An additional expenditure of $1540 was required to move and re-assemble it at the second installation. After the second pier had been completed to a safe height and the cofferdam removed, the procedure at Huffman for completing the cross dam and conduits was practically the same as has already been described in connection with the work at Taylorsville. The original screening and mixing plant was still available for this portion of the work, and the mixed concrete was run down about the same route as before by elevating the tracks with low trestles. For

handling the concrete however, a guy derrick was set up on the slope of the dam. Fig. 93 shows this part of the work under construction. At the time this picture was taken, the cofferdam for the first pier had been set in place, and the construction of the lower part of that pier was nearly finished.

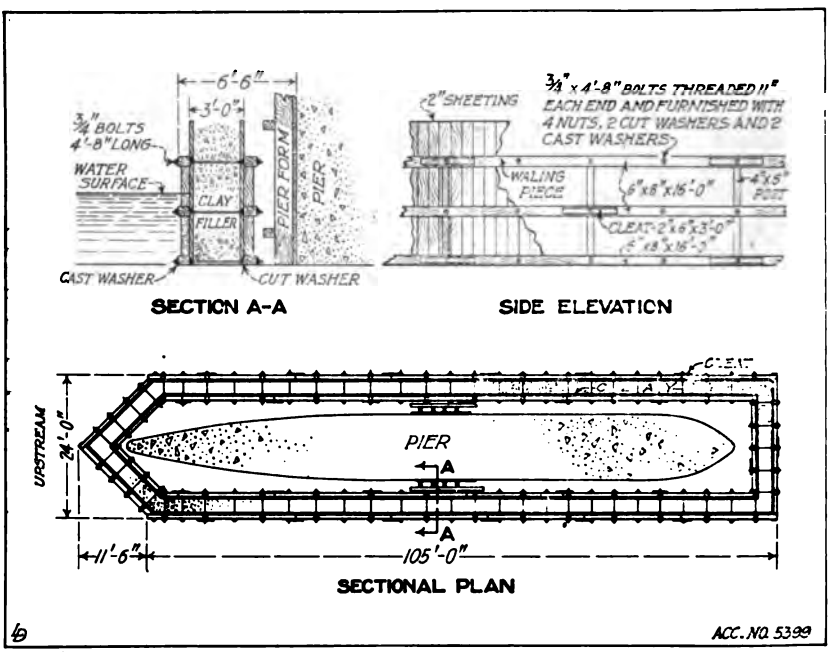


FIG. 92—DETAILS OF PIER COFFERDAM AT HUFFMAN

The piers had to be started at floor level, requiring a carefully constructed cofferdam as normal low water stood about 3½ feet deep on the floor.

Costs at Huffman: The total quantity of Huffman concrete was 37,250 cu. yds. which was placed at an average cost of $9.83 per cu. yd. Following is the final cost report for that work:

Unit Cost of Concrete—Huffman Dam
(37,250 Cu. Yds.)

		Cost per Cu. Yd.	Percent of Total
Superintendence and Overhead	$ 14,352.49	$0.38	3.9
Labor	143,826.06	3.86	39.3
Cement at $2.27 per barrel	118,380.50	3.18	32.3

Supplies (including form lumber, coal, electric power, etc.)	66,625.94	1.79	18.2
Plant Depreciation (including installation and dismantling)	23,174.40	.62	6.3
	$366,359.39	$9.83 per cu. yd.	

The wage scale was about the same as at Taylorsville, and so was cost of cement and lumber. Because of the poor grading of the sand, the amount of cement used per cubic yard of concrete was 1.4 barrels, as against 1.1 barrels at Taylorsville and 1.2 at Lockington.

An analysis of the same unit cost on the basis of materials, forms, mixing and placing is as follows:

	Cost per Cu. Yd. of Concrete in Place	Per Cent of Total
Cement (1.4 bbls. per cu. yd.) including loss on sacks, cost of unloading and storage, and cost of hydrated lime)	$3.53	35.9
Forms	2.23	22.7
Sand and Gravel in bins	1.34	13.6
Mixing	0.74	7.5
Placing	1.99	20.3
	$9.83	100.0%

The above includes a proper distribution of overhead, plant charge, etc.

The best month's record at Huffman was 6,060 cu. yds., but the output was fairly steady from the time the work started, and the work was so arranged that no concrete had to be laid in freezing weather.

Procedure at Lockington: The low water flow of the river at Lockington was much less than at the other dams, and stood less than a foot deep on the floor of the conduit structure. For that reason it was feasible to bring up the lower section of the pier between the conduits to above water elevation and still offer very little obstruction to flood discharge through the gap. This was done, bringing up the base of the pier about a foot above floor level at the time the walls and floor were built, and when the completion of the pier was started, at a later date, the pier forms were built tight around this base and were made sufficiently water-tight so that no cofferdam was required. The

water was turned through the gap as at the other dams, and the river was thus controlled during construction until the hydraulic fill was nearly completed. Then the cross dam along the spillway and the two outlet conduits were finished as was done at the other places. The original gravel plant was still available to furnish material for this work, and the concrete was carried to the work on a track raised up on trestles as was done at Huffman. One of the guy derricks, set up on a raised platform clear of the water, lifted the mixed concrete into place in the forms.

Costs at Lockington: The total quantity in the Lockington structure was 31,650 cu. yds., and it was built at a cost of $9.90 per cu. yd. Following are the cost figures. It is worthy of notice that the unit costs at these three dams were so nearly the same, the widest difference being less than 4 per cent, and two of them being within ¾ of 1 per cent of each other.

Cost of Concrete Outlet and Spillway Structure at Lockington Dam
(31,650 Cu. Yds.)

		Cost per Cu. Yd.	Percent of Total
Superintendence and Overhead	$ 10,625.87	$0.34	3.4
Labor	131,935.48	4.16	42.1
Cement at $2.25 per barrel	85,455.00	2.70	27.3
Supplies (including form lumber, coal, electric power, etc.)	54,248.14	1.72	17.4
Plant Depreciation (including installation and dismantling)	30,909.70	.97	9.8
	$313,174.19	$9.89	

The labor and material costs were practically the same as at Taylorsville and Huffman. The amount of cement used here, per cubic yard of concrete, was 1.2 bbls.

Grouting Foundations at Lockington: The Springfield limestone which formed the foundation for the structure at Lockington was quite different from the Cincinnatian rock at the other dams. This limestone, while solid and hard in itself, lay in 4-in. to 12-in. strata near the surface, and was more or less seamy, and some of the seams were open to the extent that seepage and upward pressure might occur from water backed up in the basin. To prevent this an extensive program of grouting and drainage was carried out.

Test pit exploration indicated that at a depth of from 15 to 20 feet there were continuous strata of firm hard limestone, 20

inches or more in thickness, with tight seams. The grouting operations consisted of three lines of 2-in. grout holes across the site of the structure, along the center line of the dam and extending well beyond the outside of the walls on either side. Connecting with these at either end, two lines of grout holes were extended downstream on either side to the end of the stilling pool. This was to prevent any possible upward pressure under the floor of the pool.

FIG. 93—CONSTRUCTION OF PIERS AT HUFFMAN
The original mixing plant was used and the guy derrick on the right handled the concrete. This view shows the lower part of the first pier nearly finished.

Before the grouting was commenced, the holes were thoroughly washed by flushing them with running water under slight pressure. The grouting was done by a single drum Ransome grouting machine operated by compressed air. Fig. 94 shows these grouting operations under way. The grout was mixed in the machine and then forced into the hole by means of compressed air, through the line of 2-in. hose, which was connected to a piece of 2-in. pipe previously cemented into the top of the hole to be grouted. For reasonably tight holes a thin mixture

composed of about one part cement and five parts water was used. For the more open holes the grouting was started with a mixture as thick as four parts cement, two parts fine sand, and three parts water, but in finishing off all holes the thinner mixture was used. It was necessary to use comparatively low pressure, in order to avoid danger of lifting the rock. Great care was used to keep the flow of grout continuous, by closing the valve at the machine before all the grout had been forced out of the connecting hose. In laying out the grouting program it was realized that the most desirable procedure was to drill and grout one hole before drilling another. For various reasons however, this was not practicable, and the plan was adopted of drilling a series of holes within a certain limited area, and after grouting them to drill a few test holes among them, and test for leakage. This proved very satisfactory, as practically all of the test holes indicated that a thorough job of grouting had been done. In all, 146 of these holes were drilled, to an average depth of about 20 feet, and 314 barrels of cement were used in the grouting operations.

As a matter of further precaution against any possible accumulation of upward pressure, a series of drain holes, 30 in number, were drilled to a depth of 20 feet into the rock through the concrete in the floor of the stilling pool. These holes were filled with gravel of a uniform diameter of about ¾ inch. Also a line of drain holes was drilled across the throat of the structure just downstream from the grouted holes, and these were vented by means of vitrified pipe laid in confined gravel underneath the concrete floor, and brought into the discharge channel near the upper end of the stilling pool.

The drilling of the 146 grout holes and 42 drain holes, totaled 3390 feet of drilling, (average depth 18 ft.) and cost $2390, or 71c per foot. The labor cost of grouting 146 holes, was $635, and the cost of the 314 bbls. of cement used was $690, or a total of $1325 for the grouting, not including overhead or plant depreciation.

Spillways at Germantown and Englewood Dams

At these two dams, where the culvert type of conduits was used, the spillways necessarily were separate structures. The one at Germantown was simply a gap, lined with concrete, through a depression in the hill about 800 feet north of the dam.

The bottom of the trench was in rock, 75 feet wide at the bottom, and was carried to a depth of 15 feet below the top of the dam. The concrete lining, or paving, on the slopes, extended for a distance of about 150 feet, but across the bottom at about the center were simply two concrete cut-offs about 5 feet deep, for the purpose of holding the bottom to the desired elevation, against scouring in case of flood. There were only about 500 cubic yards of concrete required for this work, and the most interesting thing about it is its simplicity.

FIG. 94—GROUTING LIMESTONE FOUNDATION AT LOCKINGTON

Before grouting was commenced, the holes were flushed with running water. Grouting was done with a single drum Ransome grouting machine operated by compressed air. There were three lines of 2-in. grout holes across the site of the outlet structure and two lines at either end extending downstream to the end of the stilling pool.

The Englewood spillway was a more elaborate structure because the rock there was at too low an elevation to form the crest of the spillway, and therefore an overflow weir was required on top of the rock to bring the crest to the required height. This weir was flanked on either side by retaining walls, completing the gap through the dam. The crest of the weir was 100 feet long and its elevation was 16 feet lower than that of the top of the dam. The highest of the retaining walls was about 60 feet high, and there was a paved stepped floor and

training walls extending about 300 feet downstream from the weir. This structure contained 7,830 cubic yards of concrete. Its foundation was in Cincinnatian rock.

The methods of construction were similar to those used in building the outlet structures at Taylorsville, Huffman and Lockington Dams, which were somewhat similar types to this, although they were considerably larger and were much more complicated because of their river control problems. Sectional wall forms were used similar to those used at the other three structures mentioned, and here, as at the other places, a lean mix of concrete was used throughout, except in the exposed faces of walls and floor.

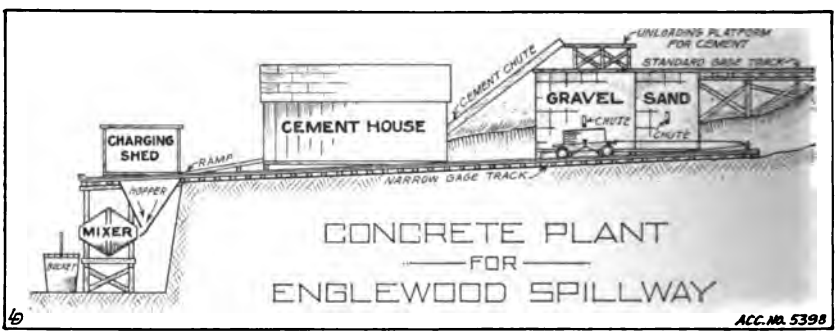


FIG. 95—CONCRETE PLANT FOR ENGLEWOOD SPILLWAY

The concrete plant for this spillway was somewhat unusual and is illustrated in Fig. 95. The main washing and screening plant, which was set up for the construction of the conduits, was available at the time the spillway was being built, and furnished the sand and gravel for this work. This washed and screened material was loaded into 12-yard standard gage dump cars, and hauled up to the site of the spillway where the material was dumped off a raised track into temporary side hill bins. As most of this trackage was a part of the regular track system on the job, it did not involve much additional expense. The bins discharged by gravity, through properly controlled openings, into a specially built car which held the proper quantity of sand and gravel in separate compartments. This car ran by gravity

on an inclined narrow gage track, along the face of the bins, sloping down to the charging hopper over the mixer, and was hauled back empty by a back-haul cable operated by a small hoist set up at the mixing plant for that purpose. A cement shed of suitable capacity was set up at the end of the elevated standard gage track, in such position that cement could be unloaded directly into it from the cars and fed from it down to the mixer platform, as needed, by means of chutes. A one cubic yard

FIG. 96—CONSTRUCTION OF ENGLEWOOD SPILLWAY
The stepped floor is in the foreground. The two retaining walls are nearly finished, but the weir has not been started.

Smith mixer was installed for the purpose of mixing the concrete, and the material was placed in the forms by means of a steel guy derrick with 105-ft. boom, handling bottom dump buckets such as were used at several of the other dams.

Fig. 96 shows this work under construction. The stepped floor is in the foreground. The two retaining walls are nearly finished but the weir itself has not yet been started. The mixing plant and cement house, and a portion of the elevated standard gage track, may be seen in the rear. The 7,840 cu. yds. of concrete in this structure were all placed within a period of six

months, the best month's work being 2,540 cu. yds. Following is the cost record:

Cost of Concrete Spillway at Englewood Dam
(7,840 Cu. Yds.)

		Cost Per Cu. Yd.	Percent of Total
Superintendence and Overhead	$ 4,724.52	$0.60	4.5
Labor	48,746.30	6.22	46.5
Cement at $2.26 per barrel	26,577.60	3.39	25.4
Supplies (including form lumber, coal, electric power, etc.)	15,585.25	1.99	14.8
Plant Depreciation (including installation and dismantling)	9,147.26	1.17	8.8
	$104,780.93	$13.37 per cu. yd.	

On this job common labor was paid 40c to 42c per hour, carpenters 75c per hour, carpenter helpers 42c per hour, all men on an hourly basis being given 11 hours' pay for 10 hours' work (time and a half after 8 hours) which meant increasing the above rates by 10%. Cement cost $2.26 per barrel including loss on sacks, and form lumber was worth about $35 per M. ft. B.M. On this structure 1½ barrels of cement were used per cubic yard of concrete.

Slope Paving

Description. In connection with the river channel improvements through the cities, paving was used on the slopes on the outsides of curves, and at other places where velocities might be high, or where for any other reason something more than sod protection of the slopes might be required. The plan was adopted of using a monolithic slab paving on the steeper levee slopes, and a flexible block revetment on the flatter beach slopes inside the toes of the levees.

Fig. 97 shows the details of this paving. A monolithic revetment is laid in slabs 8 feet wide, and where the length up and down the slope is more than about 13 feet, a horizontal expansion joint is introduced. This paving is 6 inches thick, reinforced with wire mesh, and at the toe of the slope there is a footing about 2 feet wide and 18 inches deep. This caps a row of timber piles about 8 feet long which are driven along the toe of the slope about 3 feet apart. The flexible block revetment is made up of individual blocks 5 inches thick, 12 inches wide, and 2 feet long, which are lightly reinforced with No. 7 gage

steel wire bent as shown. The blocks are laid with the long dimension up and down stream. Each block is pierced with two ¾-in. holes so placed that when the blocks are laid in adjacent rows with staggered joints, the holes line up as shown on the drawing, permitting a length of ½-in. galvanized wire strand to be threaded through them as shown, binding the whole mat together. This wire rope is anchored into the foot-

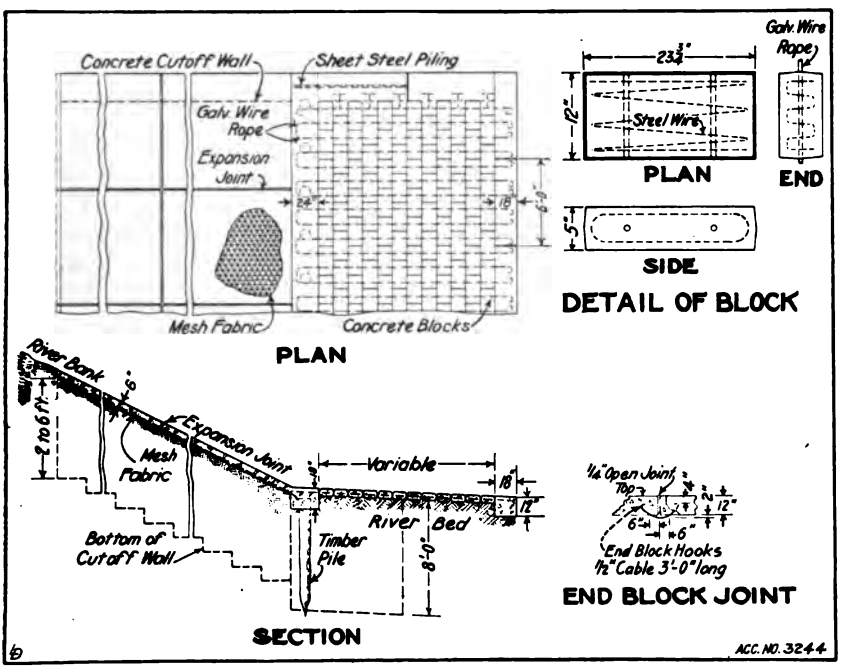


FIG. 97—DETAILS OF CONCRETE SLOPE PAVING ON LEVEES

The monolithic revetment is reinforced with mesh fabric and is laid in slabs 8 feet wide. The flexible blocks are fastened together with ½-in. galvanized strand, making a mat.

ing wall at the toe of the levee slope and is held at the outer end by being cast into the heavy section of blocks which bind the outer edge of the revetment, the idea being that in case there is any cutting and the outer edge is undermined, the blocks will drop down, following the cutting, and act as a curtain to prevent possibility of undermining the toe of the monolithic pavement. The heavier outer blocks are held together by a short

piece of wire strand, anchored into the end of each block, and passing under the joint as shown on the drawing. The width of the flexible mat varied in different places from 10 to 35 feet. It was found by experience, however, that a 10 foot strip was not wide enough. It is better to fix the minimum width at 15 feet.

At the upstream edge of each section of slope paving and at intervals of about 300 feet, a concrete cut-off is built as indicated on the drawing, and this is continued out under the edge of the flexible block revetment for half its width, by a curtain of steel sheet piling.

Making the Blocks. Blocks for the flexible revetment were manufactured under contract, the plant being located at a convenient place along the bank of the river where sufficient storage space was available. Material for the aggregate was thrown up on the river bank by a dragline machine during the course of channel excavation. A screening plant was installed with a crusher to reduce the over-size stones, as the size of aggregate was limited to what would pass a 2-in. screen. A traveling stiff-leg derrick fed raw aggregate from the pile to the crusher; a hopper car shuttled back and forth from derrick to crusher when the derrick was out of reach. From the screening plant bins, a partitioned hopper car carried charges to the mixer. Fig. 98 is a plan showing the plant layout, and Fig. 99 is an enlarged floor plan of the mixer house.

The blocks were cast on cars large enough to hold sixteen blocks lying flat. Fig. 100 shows a car with the complete form arrangement. The car floors were covered with metal and served as forms for one side of the blocks. End and side forms were made by longitudinal and cross pieces as shown in the photograph. These pieces were faced with 16-gauge sheet steel, shaped to give the required bevel on the edges of the blocks, and were all removable except the cross partition in the middle of the car. All cross pieces were interchangeable, as were all longitudinals. Each short piece was made with two tenons which fitted into mortises in the cross pieces, and after all parts were assembled the whole was made rigid by wedges bearing against the posts shown at the ends of the car. All movable parts could be used with either edge up or could be turned end for end, a feature which greatly simplified handling and assembling. Holes through the blocks were formed by iron rods slipped

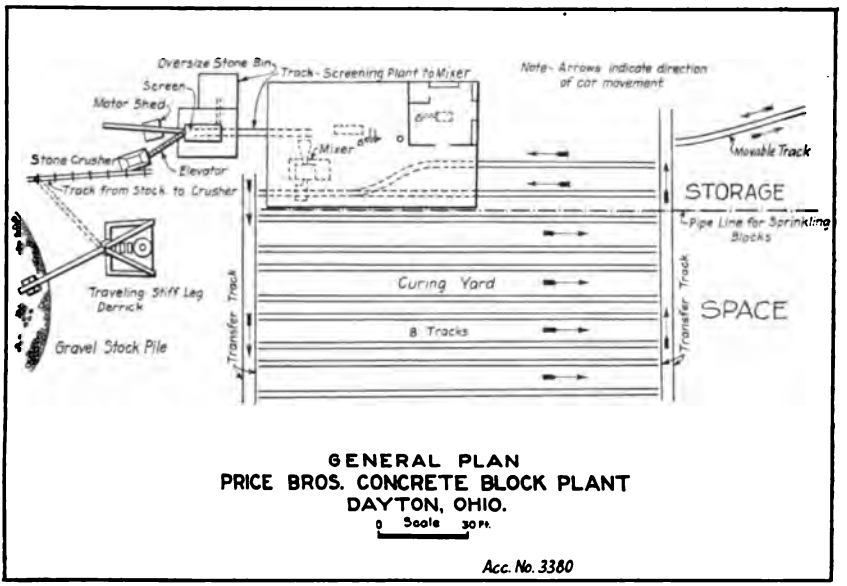


FIG. 98—LAYOUT OF CONCRETE BLOCK PLANT

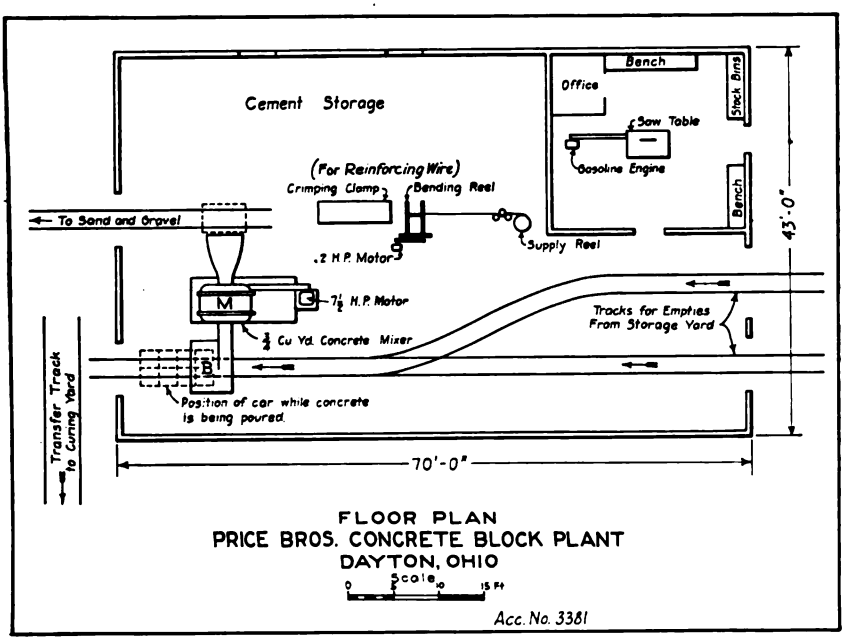


FIG. 99—FLOOR PLAN OF MIXER HOUSE IN CONCRETE BLOCK PLANT

through holes in the forms as shown. After each usage, forms, car, platform, and rods were thoroughly cleaned, and painted with straw paraffin oil. Rods were turned in place after the concrete had set very slightly and were removed while it was still rather green. After 24 hours on this car in a curing yard, blocks were removed by knocking out the end wedges and taking the forms apart.

Reinforcement was of No. 7 steel wire bent as shown in the nearest block cell in Fig. 100. It was a considerable problem to properly shape the reinforcement but a satisfactory scheme was evolved. The layout is shown on Fig. 99. As the wire came from a reel it passed first between a set of three grooved wheels, 5 inches in diameter, arranged with one opposite the other two, and close enough together to give the wire a rather abrupt reverse curve as it fed through. This eliminated the bends due to the original coiling. The wire was then wound on a revolving reel made with two parallel pipes so set as to bend the wire into loops of the required size. Then, the bends were crimped more definitely into the required shape by a large machine which clamped a whole reel of wire at one time between heavy metal lined jaws. This at the same time flattened the loops and loosened them from the pipes sufficiently to permit easy removal, after which the wire was cut into pieces having the requisite number of loops.

Concrete from the mixer was discharged into an inclined flat box under which the cars were wheeled. Four gates in the lower side of the box allowed the concrete to discharge simultaneously, if desired, into four of the block forms, and by slowly moving a car ahead under the box a full pour was completed very rapidly. While pouring concrete, the reinforcement was first allowed to rest on the rods as shown in Fig. 100, and after the forms were partly filled it was lifted into place. As soon as a car was filled it was removed to the curing yard, where the concrete was thoroughly spaded and the top given a rough finish with a trowel. Curing for 24 hours made the blocks hard enough to be safely removed from the car, after which they were piled in a storage yard and sprinkled for several days, the period depending upon atmospheric conditions.

The total crew was 24 to 28 men. After reaching full production, an average day's run was 1300 blocks. The total number made for the Dayton work, at the plant just described, was about

185,000 blocks. The plant was then moved to Hamilton where 230,000 blocks were made for the work there. Under the terms of the contract the District furnished cement, sand and gravel, and steel; the contractor furnished all labor and equipment. The mix of concrete used in the blocks was about 1:2:4 with the gravel limited in size to what would pass a 2-in. screen. The contract price was 20c per block for the first 100,000 turned out from each setting of the plant, and 16c per block for additional quantities. The total cost to the District for the blocks

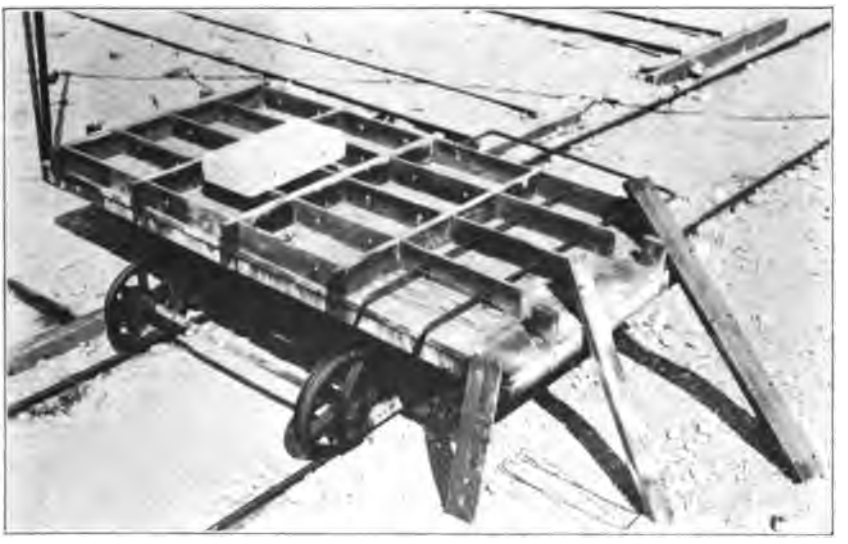


FIG. 100—CAR WITH FORMS FOR CONCRETE BLOCKS

Sixteen blocks could be cast on one car. The longitudinal and cross pieces of the forms were of 16-gage sheet metal and were interchangeable. Holes through the blocks were formed by iron rods slipped through the holes in the forms.

made at Dayton, including materials and inspection, was 33.5c per block in the storage pile. The cost at Hamilton was slightly less, due to the larger quantity.

Pile Driving: The first step in laying the slope paving was the driving of the row of piles along the toe of the steep slope. At Dayton these piles, 8 feet long, were furnished and driven at a contract price of $3.80 per pile in place. At Hamilton the District furnished the piles, and they were driven at a contract price of $3.00 per pile in place. The contract price for driving

steel sheet piling was 35c per square foot in place; the District furnished the piling.

Laying Flexible Block Revetment: Following the pile driving, the levee and channel slopes were carefully dressed to the required grades, and then followed the laying of the blocks. Here, as in the block manufacture, study and experiments were necessary before developing satisfactory methods.

It was no mean job to haul the blocks from storage to the river bottom. For this work, specially built wagons with underslung platform bodies were used, which facilitated loading and unloading. The blocks weighed about 120 pounds apiece and 45 made a good load under most conditions. Three or four wagons could haul blocks as fast as required, each driver doing all handling for his own hauling and distributing the blocks along the site of the work in advance of laying. The largest day's work by one team was the rather astonishing number of 700 blocks.

The ½-in. galvanized strand, on which the blocks were strung, was delivered on reels containing 5000 feet each. The first step toward laying the blocks was the unreeling of cable and cutting it into suitable lengths. The cable was rather hard drawn and as it was unwound, it tended to retain the curve of the reel, the curvature causing it to bind against the sides of the holes in a very aggravating manner as it was drawn through the blocks. The curvature was removed by a method exactly similar to that employed for straightening the reinforcing wire for block manufacture. As it came off the reel it was fed between three grooved wheels of 8-inch diameter, staggered so as to give a reverse curve sufficient to take out the original set. Various methods were tried of cutting the wire so that the strands would not flare and make the ends larger than the holes in the blocks. Neither ordinary shearing, nor burning off, gave satisfactory results. It was eventually cut by a shearing machine with small arcs ground opposite each other in the jaws. The result was a compacted end which readily entered the holes.

Blocks were laid a row at a time, beginning with the row nearest the levee. The cables were laid loosely in place on the levee slope, and as soon as the first row of blocks was laid the ends of the cables were threaded through the blocks and about 2 feet into the clear. The next row was then laid, breaking joints with the first one, each block being slipped over two

adjacent projecting ends of cable, and the cables pulled through as before. The process was repeated until the full width of revetment had been laid, after which there was sufficient cable extending to anchor the blocks to the footing wall and end blocks, which were later cast in place. The procedure is well illustrated by Fig. 101. The crew consisted of four men. One, seated on the blocks brushed away any surplus matter from the bed for the new block to be laid, and entered the ends of

FIG. 101—LAYING CONCRETE BLOCKS IN REVETMENT
A four-man crew handled this job.

the cables in the holes of the block as it was brought into position by two other men. The men carrying the block handled it with specially made tongs as seen in the photograph. The fourth man followed, and pulled the cable through in readiness for the next row. In the beginning he pulled the cables by hand, but even slight irregularities in the alignment of the blocks caused them to bind in the holes and a special tool was prepared for the job. A wooden handle, five feet long, was hinged to

one end of a 2-inch plank about two feet long and a pair of "come-along" tongs was attached to the handle a foot from the hinge. In operation, the free end of the plank was placed against a block under the cable to be pulled and with the handle pointing up. The operator tipped the handle toward the blocks, gripped a cable with the tongs, and pulled the cable through very easily. The four men laid about 800 blocks per day as an average, although by working extra hours, as many as 1400 blocks were laid in one shift.

Placing Monolithic Slab Paving: Various types of equipment were employed from time to time, for placing the monolithic slabs on the levee slopes, the final development being steel forms and a portable mixing equipment. The side forms were made of 6-inch channel irons with plates at the bottom shaped to fit the footing walls; the top was a 6-inch plate; and the bottom was held by an angle bar over the edge of the blocks in the flexible paving. Corner connections were made by tenons on the upper and lower members which extended through mortises in the side forms, the joints being locked by steel wedges driven through slots in the tenons. Alternate slabs were poured with the forms, and the intermediates were poured while the forms were being used elsewhere. See Fig. 102.

The mixer was a No. 7-L Jaeger with some alterations. It was mounted on a motor truck chassis and so arranged that the driving mechanism could be geared direct to the automobile engine. The guide tracks for the loading hopper were extended to permit the hopper to rest on the ground. When it was desired to move, the mixer was thrown out of connection and the truck operated as usual. Extending out from one side and up the levee slope was an I-beam boom along which could be pulled a bottom dump bucket. In operation, the end of the boom was supported by a small wooden A frame, and after the bucket had received a charge from the mixer it was pulled (also by the truck engine) to whatever position was desired along the boom and dumped on the slope. In some instances, however, the mixer was operated on the top of the slope and the concrete fed down to place by chutes. Concrete was spread in the forms by hand and thoroughly tamped. The wire mesh reinforcement was first placed on the earth slope in the forms and as soon as part of the concrete had been placed, the mesh was lifted by hand to its proper location. Cement, sand and gravel were distributed

along the line of work by teams. Water was usually pumped from the river by a portable gasoline pumping equipment.

The concrete was mixed in proportions about 1:2½:5, of rather dry consistency, and each slab was faced with a thin coat of mortar slightly colored with lamp black, to relieve the glare from finished work. The concrete was well tamped into place and was carefully cured by sprinkling for several days after being placed. The crew, not including teamsters for hauling, was about 20 men. Under fair conditions 12 slabs constituted a day's

FIG 102—POURING MONOLITHIC REVETMENT SLABS

The mixer was mounted on a motor truck chassis, with the driving mechanism geared direct to the motor. The side forms were 6-in. channels with plates at the bottom shaped to fit the footing walls. Alternate slabs were poured.

work, the largest number for one day being 16, which, with the accompanying footing blocks, amounted to about 54 cubic yards.

Following are some figures showing the cost to the District of the paving blocks, the laying of the flexible block revetment, and the slope paving in place. All of this work was done by contract, the District furnishing all materials and the contractor furnishing all labor, supplies, and plant. The contract for the manufacture of blocks was on the basis of a fixed price per block, as has been explained. The contracts for laying blocks and for the monolithic slab revetment were variable fee contracts, by the terms of which the contractor's base fee was increased or de-

creased according to the ratio of estimated cost (agreed to in advance) to actual cost. This form of contract is explained more fully in Part IX* of these Technical Reports. In both these cases the actual cost to the District, including the contractor's fee, was less than the estimated cost as follows:

Laying block revetment, estimated cost 15½c per block.
Actual cost, 13.2c per block.

Placing slab revetment, estimated cost $8.25 per cu. yd.
Actual cost, $6.92 per cu. yd.

Costs: The detailed costs of these operations are as follows:

Cost of Flexible Block Revetment—Dayton

Cost of blocks per cu. yd. (Total quantity 185,000 blocks)

Cement	$3.23	
No. 7 Wire (reinforcement)	1.10	
Gravel	0.40	
Contract Price of Manufacture	5.90	(18.2c per block, average)
Inspection	0.25	
	$10.88	per cu. yd. in storage

There were 32½ blocks per cu. yd., making this cost 33½c per block.

Cost of Hauling and Laying Per Cu. Yd.

½-in. Galvanized strand (71.3 lin. ft. @ 3c)	$2.14	
Contract price for hauling and laying	4.28	(13.2c per block; estimated cost 15½c)
Inspection	$0.20	
	$6.62	per cu. yd.

Total cost of Flexible Block Revetment in place, $19.88 for blocks; $6.62 for laying; total $17.50 per cu. yd. in place.

Cost of Monolithic Slab Revetment—Dayton

Total Quantity—7468 Cu. Yds.

	Cost per Cu. Yd.	
Cement	$ 3.76	
Sand and Gravel	1.87	
Wire Mesh (54 sq. ft. @ 1.7c)	0.92	
Expansion Joint Material (12 lin. ft. @ 2c)	0.24	
Contract Price for Mixing and Placing (Including plant & forms)	6.92	(Estimated cost $8.25)
Inspection	0.25	
	$13.96	per cu. yd. in place.

*Technical Reports, Part IX. Accounting and Cost Keeping.

Gravel Barrier Dam at Hamilton

Mention has been made in Chapter II of a low concrete dam built at the upper end of the work at Hamilton to act as a barrier against the drift of gravel down the river channel, the excavation and pile driving for this structure being handled by a dragline excavator. (See Fig. 15.)

The dam is of the overflow type and rests on a porous foundation of well graded gravel, varying in size from ordinary sand to boulders of more than a foot in diameter. The length between abutments is 800 feet, and the height above the stream bed is from 5 to 9 feet. The standard cross section of the dam is shown in Fig. 103. The crest is level all the way across the river, and the profile across the apron is a level line also, except in the

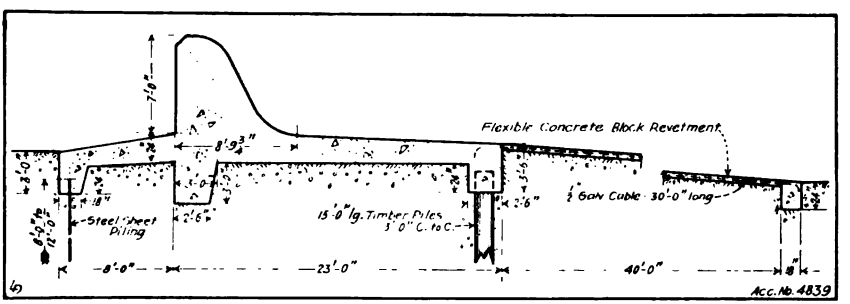


FIG. 103—CROSS SECTION OF BARRIER DAM AT HAMILTON

deepest part of the channel where the apron is dropped to fit the channel bottom; the body of the dam across that section is somewhat deeper than shown. A continuous cut-off wall of sheet piling, from 8 to 12 feet in depth, is provided along the edge of the upstream apron, and a row of timber piles 3 feet apart is provided as shown along the downstream edge of the main concrete apron. An extension of this concrete apron was provided in the form of flexible block revetment such as was used to protect the slope paving along the river channel. It was found, however, that this block revetment was not satisfactory in this place, and it was replaced later with a heavier apron as will be explained.

The construction of the dam was taken up in two sections, one-half its length being built first, except that five sections of the

body of the dam, each 24 feet long, were left out to form gaps for the diversion of the river while the other half of the dam was being built. This is illustrated in Fig. 104. Sand and gravel for concrete were obtained from a commercial plant which happened to be located just above the east abutment of the dam. Incidentally, this plant makes use of the gravel which drifts into the shallow basin above the dam, and its operations keep the basin open to receive the gravel which is brought down to that point during times of high water. The materials were delivered from this gravel plant to a one cubic yard mixer, by a 1½-yard

FIG. 104—PARTLY COMPLETED BARRIER DAM AT HAMILTON
The dam was built in two sections, one-half its length being built first, except that five sections, each 24 feet long, were left out to form gaps for the diversion of the river while the other half of the dam was being built. Concrete was delivered over the trestle just above the dam.

rocker dump car on a gravity track handled by a small electric hoist. For the purpose of delivering the concrete to the dam, a trestle was built just above the face of the dam, the top of the trestle being about 2 feet above the crest of the dam. This is also shown in Fig. 104. The deck of the trestle was extended as required by the progress of the concreting operations. A side hopper dump car, hauled by a 3-ton gasoline locomotive, transported the concrete from the mixer to the work. The dam was built in 30-ft. sections, except where the 24-ft. openings were left, thus providing contraction joints at suitable intervals.

After the dam was completed, with the exception of the five openings, the far section of the trestle was removed and the open-

ings were filled one at a time, beginning with the one farthest from the shore. Each opening was closed by a cofferdam of 8 x 16 timbers offset far enough from the face of the dam to leave room for building forms. These cofferdams rested on the upstream concrete apron and were made practically watertight. It happened that when the last opening was to be closed, almost the entire flow of the river was being used by a power plant, which by-passed the water around the dam, so it was possible to set up forms for this final closure, pour the concrete, allow it to set sufficiently to enable forms to be removed, and to remove the forms, before the basin above the dam was filled. It was 60 hours from the time the last closure was made until the water rose to the top of the dam, and with this very small flow, it was a simple matter to protect this last section of concrete until final hardening had taken place.

While the use of the flexible block revetment along the channel slopes has been an unqualified success, as evidenced by experience during several floods, it was found that the apron of this dam was too light to stand the effect of the disturbance from the overflow. After the first high water, this part of the apron was badly disturbed, and apparently had undulated from the effects of the current like the waving of a flag in a breeze. It was necessary to reinforce this part of the apron by removing some of the blocks around 8 foot squares, and adding to the thickness and weight by excavating underneath and reinforcing with additional concrete. In some cases it was easier to substitute heavy precast blocks about 1½ to 2 ft. cubes. (There was no rock available, suitable for heavy paving.)

At another place where flexible block construction was used similarly on a smaller scale, the same damage to the blocks occurred. Hence the conclusion that against the direct action of the current in midstream, this flexible block revetment is too light for satisfactory service. This should be no argument against it, however, as used along the channel slopes, because there, after repeated tests under various flood conditions, it has given entire satisfaction.

River Walls

At various places along the river banks, where the channel improvement work was carried on through the cities, there was not room for the regulation levee slopes without encroaching too far

on valuable property. At such places retaining walls were substituted for levees either in whole or in part. Some of these were only crest walls and assumed no great importance in the construction program. Others, however, were of considerable height and length, and called for plant and organization compar-

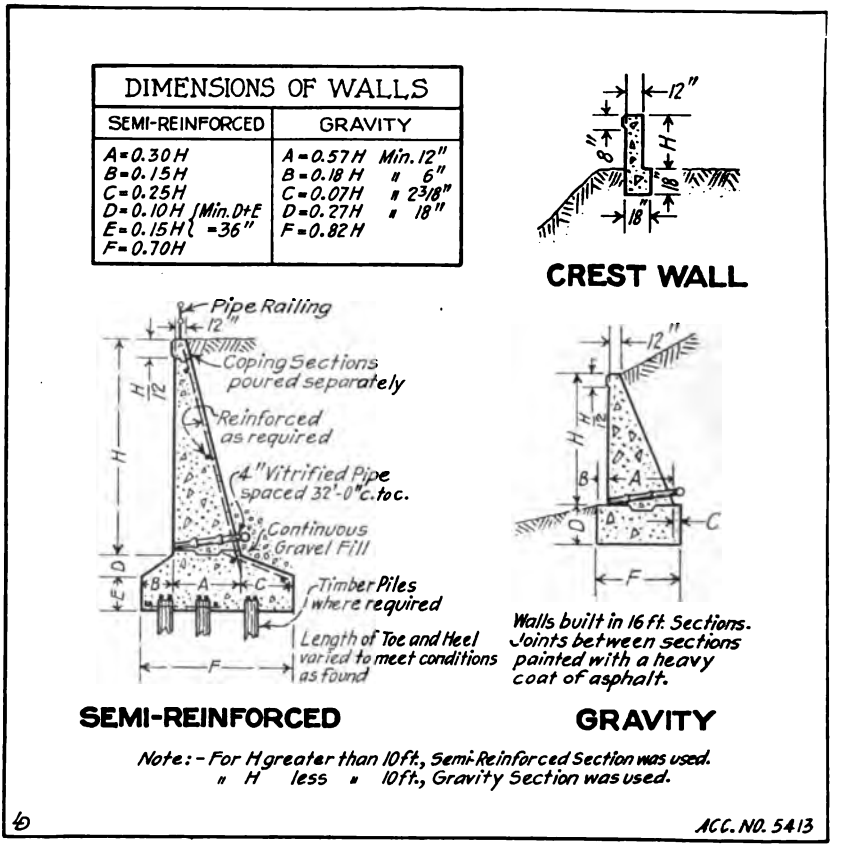


FIG. 105—STANDARD SECTIONS OF RIVER WALLS

The gravity section was usually used for walls less than 10 feet in height. Crest walls were used on tops of levees where necessary.

able with those for other important features of the work. For example, the Robert Boulevard wall at Dayton was over 1,000 feet long, had a maximum height of 25 feet, and contained 4,500 cubic yards of concrete. The Stillwater Drive wall at Dayton was 920 feet long, had a maximum height of 25½ feet, and contained

3,830 cubic yards. One of these walls at Hamilton had a maximum height of 34 feet from bottom of foundation to top of coping. This wall was 500 feet long and contained 3,060 cubic yards of concrete.

Figure 105 shows standard sections of these walls as used throughout the work. For a wall less than 10 feet in height it was customary to use the gravity section. For the higher walls the semi-reinforced section was more economical. The steel in

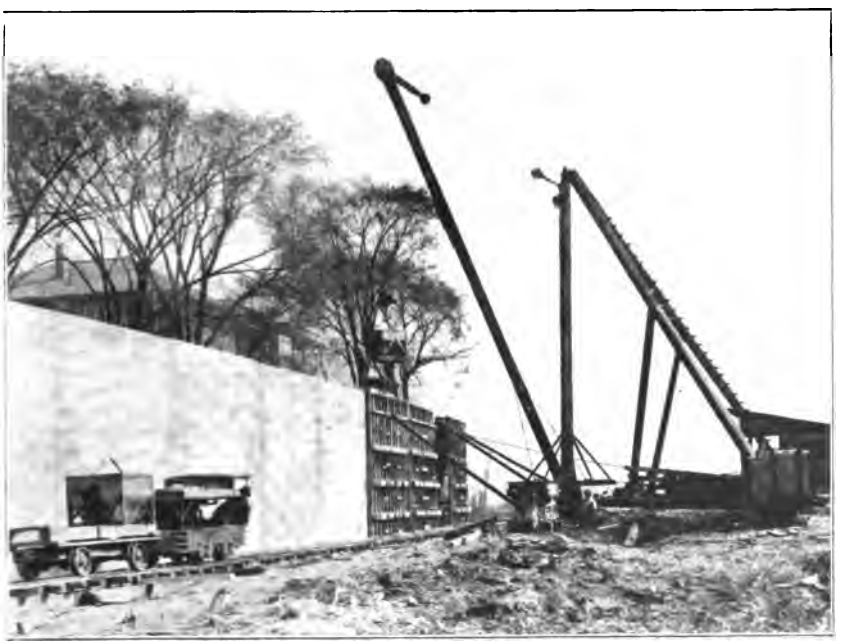


FIG. 106—CONSTRUCTION VIEW OF RIVER WALL

The stiffleg derrick was used both for handling the concrete and for setting and moving forms which were made up in standard sections and used repeatedly. In this view the coping has not yet been placed on the wall.

this section contained the equivalent of ½-in. square bars spaced 8 to 12 inches apart (depending on the load) in the shaft of the wall, 6 to 9 inches apart in the toe, and 8 to 12 inches in the heel, with only enough longitudinals to act as spacers. This amounted to 8 to 9 pounds of steel in all, per cubic yard of concrete. The walls were built in short sections, so no temperature reinforcement was required.

Wall footings were poured first and then the forms were set up for the shaft or body of the wall. At Dayton it was customary to build these walls in 16-ft. sections and to pour a section to full height at one pouring. At Hamilton the standard section was 30 feet long, and the higher walls were built in two or three lifts. Pouring the full height of the wall at once, required stronger and better braced forms, but had many advantages, such as avoiding day's work joints, helping progress, and releasing forms more

FIG. 107—COMPLETED RIVER WALL AT DAYTON

It can be seen from this view why a wall is preferable to a levee in a place like this. To get the necessary waterway the toe of the levee would have to coincide with the line of the face of this wall, throwing the levee itself over into the street or into dwellings.

quickly. Forms were built of 2-in. lumber with 4 x 6 studding and 6 x 6 or 8 x 8 longitudinals. The forms were held together by rods through the concrete (which were afterwards removed), and by bracing against the sides of the trench where convenient. It was found more satisfactory to pour the coping as a separate operation, after the shaft had been finished and the forms removed. In this way a better alignment of the top was secured. Fig. 106 shows one of these walls in Dayton under construction.

The stiff-leg derrick is used both for handling the concrete and for setting and moving forms, which are made up in standard sections and used repeatedly. It will be noticed that the coping for this wall has not yet been placed. Fig. 107 shows another one of the Dayton walls which has been completed. From a study of this view it is easy to understand why a wall is preferable to a levee in a place like this. To get the necessary waterway the toe of the levee would have to coincide practically with the line of the face of this wall. This would bring the levee itself

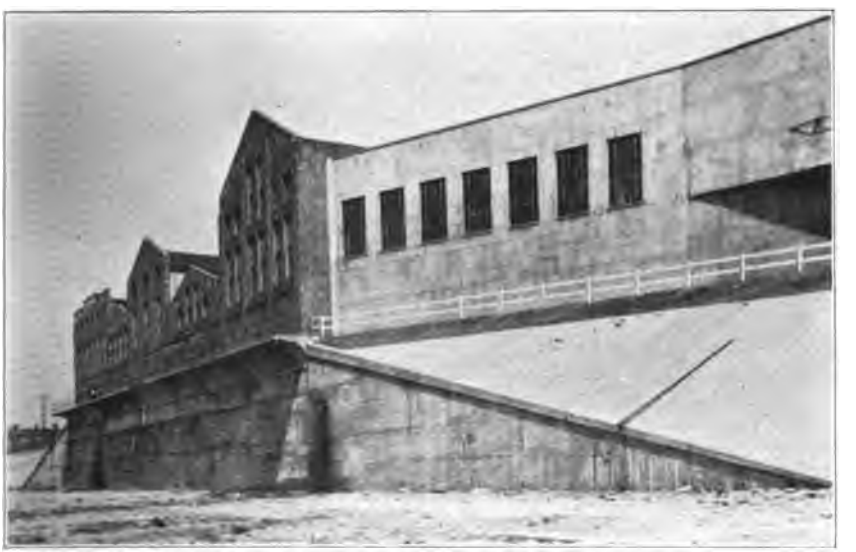


FIG. 108—WALL ALONG FACTORY BUILDING AT HAMILTON
Here the rear end of a building had to be cut off to get necessary waterway, and the wall was constructed with a cantilever walk along its top to provide a passageway around the end of the buildings. The wall is 34 feet high from bottom of foundation to sidewalk.

over into the boulevard in some places, and in others it would be necessary to acquire valuable property such as that shown here. As it was, in order to get a satisfactory alignment for the wall, it was necessary to acquire or move several dwelling houses along here.

An interesting situation at Hamilton is shown in Fig. 108. Here it was necessary to cut off the rear end of one of the factories in order to get the necessary waterway and make room for the required protection work, and the result is shown in this

picture. The wall is one of the standard semi-reinforced sections, except that it carries a cantilever walk along its top in order to provide a passageway around the end of the buildings. The construction problem here was difficult, as the work had to be done with the least possible interruption to the operations in the factory. While the construction of the wall was going on, a temporary end was put in the building far enough back to clear the construction operations. Then, when the wall was finished, the permanent end of the building was put up, using the wall

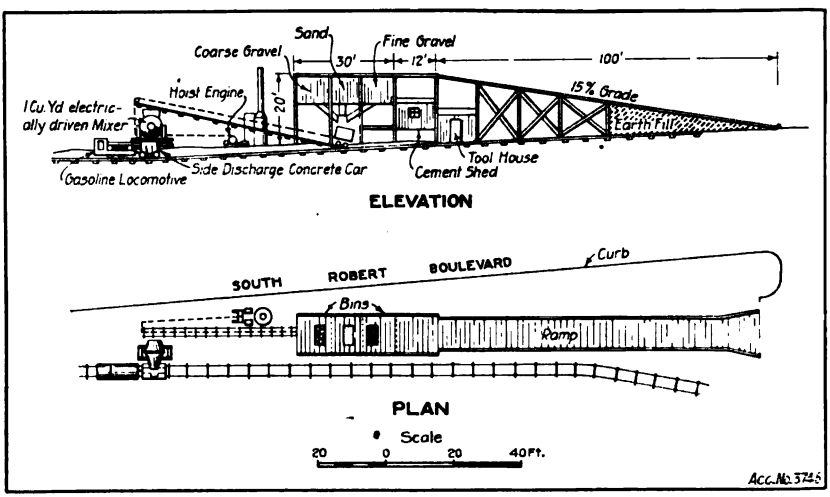


FIG. 109—TYPICAL CONCRETE PLANT FOR WALL CONSTRUCTION

Gravel was hauled in by trucks and dumped into bins. The material was fed through chutes to a batch car, cement was added, and the car hoisted up the second incline and dumped into a hopper over the mixer.

itself as a foundation. This wall is 34 feet high from the bottom of the foundation to the sidewalk.

A representative concrete plant for one of these walls is illustrated in Fig. 109. In this case the screened and washed sand and gravel was obtained from a central plant, and was brought to this mixing plant in motor trucks. The truck backed up the incline and dumped its load into the proper bin. The material was fed through chutes to a batch car, cement was added, and this car was hoisted up the second incline and dumped into a hopper, over the mixer, which held one complete batch. The plan

was to keep this hopper full at all times, and to use the material as fast as the mixer would take care of it. The mixed concrete was hauled to the work in side dump cars, or bottom dump buckets. In many places it was possible to chute the concrete into the footings or into some of the gravity section walls, but for the higher walls it was more common to use bottom dump buckets and derricks. The proportions of concrete used in these walls was approximately 1:3:6 for footings and for gravity sections, and about 1:2½:5 for the semi-reinforced walls.

The total quantity of concrete in all the walls at Dayton was 23,630 cu. yds. The average cost of this concrete was $15.45 per cu, yd., made up of the following items:

Cost of Concrete Walls at Dayton
(Total Quantity, 23,630 Cu. Yds.)

		Unit Cost
Superintendence and Overhead	$ 20,473.61	$ 0.87
Labor	165,308.46	6.99
Materials and Supplies	168,947.89	7.15
Plant Depreciation	10,448.70	0.44
	$365,178.66	$15.45

Labor rates were 40c to 46c per hour and form carpenters were paid 80c per hour. Time and a half was allowed for time over 8 hours. Most of this work was carried on in 10-hour shifts.

The above figures include extra cost of winter work, in several cases where it was essential to carry on the work through the cold weather.

Following is a detailed cost of one of these walls of the semi-reinforced type, which was 700 feet long, had a maximum height of 26 feet and contained 3,344 cu. yds. of concrete.

Cost of Concrete in Beach Avenue Wall, Dayton

			Cost per Cu. Yd.
Footings (1635 cu. yds.)			
Superintendence and Overhead		$ 890.00	$0.54
Labor—			
Forms	$3,073.63		
Mixing and Placing	2,598.98		
Shop Work	96.76		
Hauling	928.53		
Industrial Insurance	132.02	6,829.92	4.17
Materials and Supplies—			
Lumber, etc	$ 772.29		
Cement	4,519.90		
Aggregates	1,472.25		

			Cost Per Cu. Yd.
Reinforcing Steel	546.27		
Coal	79.50		
Shop	130.66	7,520.87	4.60
Plant Charge		707.25	0.43
		$15,948.04	$9.74

Sq. ft. of form surface per cu. yd.—6.3.
Cost of forms per sq. ft.—39c.

Body of Wall (1655 cu. yds.)			Cost per Cu. Yd.
Superintendence and Overhead		$ 1,445.00	$0.87
Labor—			
Forms	$3,275.30		
Mixing and Placing	3,408.72		
Shop Work	106.51		
Hauling	1,223.37		
Industrial Insurance	132.41	8,146.31	4.91
Materials and Supplies—			
Lumber, etc.	$ 502.14		
Cement	5,633.25		
Aggregates	2,070.00		
Reinforcing Steel	233.61		
Coal	1,190.00		
Shop	217.43	9,846.43	5.95
Plant Charge—			
Depreciation	$ 544.40		
Installation	2,274.49		
Dismantling	267.29	3,086.18	1.86
		$22,523.92	$13.59

Sq. ft. of form surface per cu. yd.—15.0.
Cost of forms per sq. ft. of surface—16c.

Note—Some of the form panels were transferred to this job from previous work. All new forms would have increased the unit cost of concrete about 40c.

Coping (54 cu. yds.)

The coping of this wall was placed by contract at a cost to the District of ..... $15.00 per cu. yd.

Summary

Footings	1635 cu. yds.	cost $15,948.04 =	9.74 per cu. yd.
Body of Wall	1655 " " "	22,523.92 =	13.59 " " "
Coping	54 " " "	810.00 =	15.00 " " "
	3344 cu. yds.	cost $39,281.96 =	$11.75 per cu. yd.

A similar cost record is given below on one of the walls at Dayton which was partly gravity and partly semi-reinforced type. This one was 175 feet long, its height varied from 3 feet to 23½ feet, and it contained 370 cu. yds. of concrete. Because of its constantly increasing height, and therefore its constantly varying cross section, and because of the relatively small quantity in it, this was one of the most costly of any of these walls.

Cost of Concrete in Sunset Avenue Wall, Dayton

Total Quantity, 368 Cu. Yds.

			Cost per Cu. Yd.
Superintendence and Overhead		$ 455.00	$1.23
Labor—			
Forms	$1,735.35		
Mixing and Placing	943.16		
Shop Work	44.55		
Hauling	505.72		
Industrial Insurance	72.56		
Miscellaneous	119.51	3,420.85	9.31
Materials and Supplies—			
Lumber, etc.	$ 224.68		
Cement	1,298.30		
Aggregates	316.20		
Reinforcing Steel	57.07		
Coal	247.75		
Shop	14.70	2,158.70	5.86
Plant Charge		242.67	0.66
		$6,277.22	$17.06

At Hamilton the total quantity of concrete in these walls amounted to 5,730 cu. yds. and its average cost was $15.64 per cu. yd., as shown in the following cost analysis.

Cost of Concrete Walls at Hamilton

Total Quantity, 5,730 cu. yds.

		Unit Cost
Superintendence and Overhead	$ 4,186.84	$0.73
Labor	43,657.76	7.62
Material and Supplies	40,675.62	7.09
Plant Depreciation	1,133.82	.20
	$89,654.04	$15.64

Spillway Bridges

On top of each of the dams is a highway, crossing the valley. Therefore it was necessary to have a bridge in each case to carry these highways over the spillway gaps. Figures 110 and 111 show the elevations and cross sections of these five bridges. At Englewood, Taylorsville, and Huffman, where the roads over the dams are important thoroughfares, the bridges were made 20 feet wide in the clear. At Germantown and Lockington, where the highways are of less importance, the bridges were made 18 feet wide. In all cases, the width of the roadway over the dam is approximately 30 feet. All of these are deck girder bridges, although at Taylorsville the girders are shaped to give the appearance of a flat arch. The maximum live load for all the bridges was assumed to be two 20-ton trucks traveling side by side in the same direction. The center span at Taylorsville is a cantilever span, but in all other cases there are simple girders from pier to pier.

At all of the dams except Germantown, the building of the piers for the spillway bridges was made a part of the weir construction, as that was the most economical way to handle it, and the building of the bridge itself followed immediately. At Germantown, as will be seen from the illustration, where the spillway was simply an open trench, it was more practical to build the bridge as an entirely separate job. In all cases, however, the plant for mixing and placing concrete, which was used for the building of the spillway, was available for the construction of these bridges. At Lockington, in order to make the bridge pier fit the top of the spillway weir without overhanging the upstream face, it was necessary to make the pier considerably shorter than the width of the bridge, and to meet this situation the top of the pier was corbeled out as shown in Fig. 112. This illustration shows also the details of the cross section of this bridge, which is typical of the others. The bridge railings in every case were solid paneled railings of concrete, as shown in this and some of the other illustrations.

The false work for the Germantown bridge was somewhat more expensive than that for the others, because of the greater height above the spillway bottom, but the form work there was similar to that used at the other places. Fig. 113 shows the principal details of the false work and form work for the Germantown

bridge, and Fig. 114 is a photograph of the false work and forms for the Taylorsville bridge. In each case the forms for girders and floors were built complete, and all the reinforcing steel was placed before pouring was started. Then a complete span was poured in one operation, except at Taylorsville where one-half the bridge was completed at one time.

One thing that required particular attention at all these bridges was the correct placing, and holding in place, of the

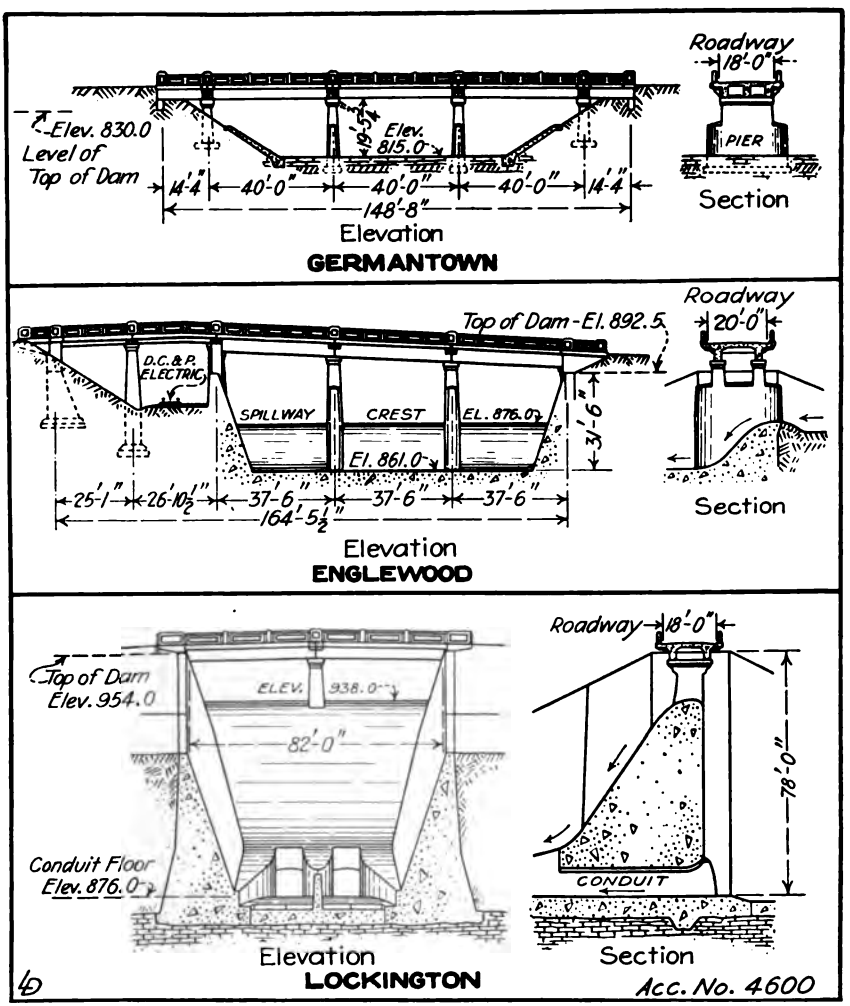


FIG. 110—ELEVATIONS AND CROSS SECTIONS OF SPILLWAY BRIDGES

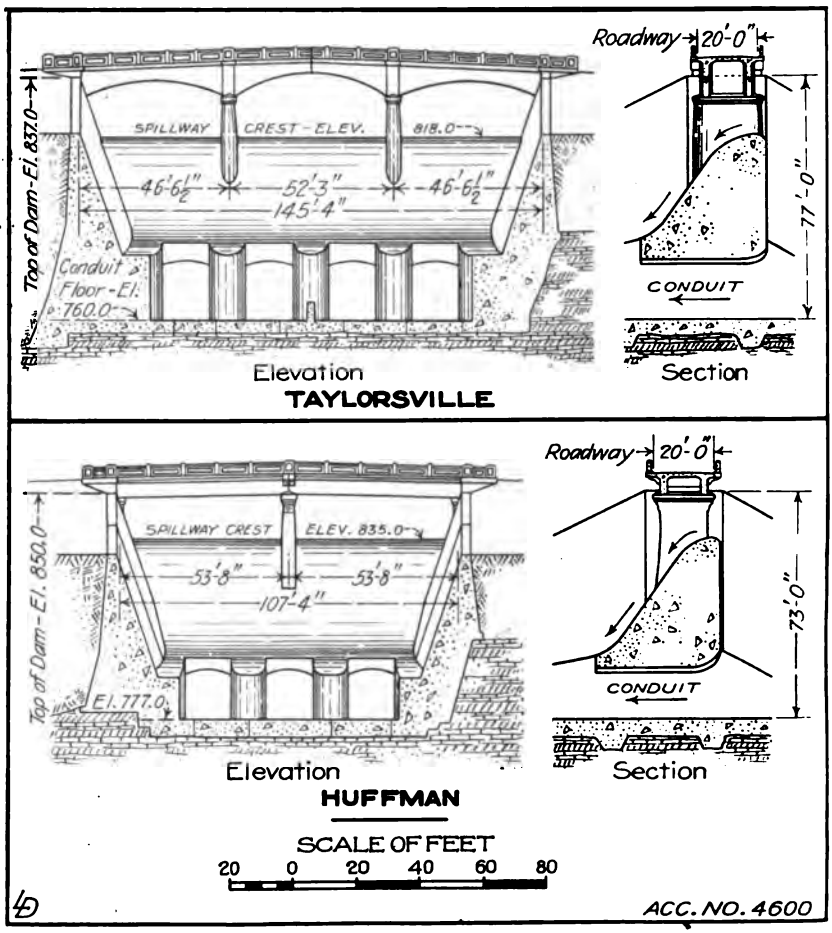


FIG. 111—ELEVATIONS AND CROSS SECTIONS OF SPILLWAY BRIDGES

reinforcing steel, as all of this work was heavily reinforced. Because of the closeness of the bars in the girders, the concrete aggregate was limited in size to about 1-in. stone, special spading tools were used, and great care was taken to secure a first-class job. In the floors, where the spacing of the steel was not so close, stones of larger size were permitted, but nothing to exceed 2½-in. material. All the concrete in these bridges was a 1:2:4 mix or richer. Fig. 115 shows the floor system for one section of the bridge at Taylorsville, with the steel set and

cleaned up ready for the placing of the concrete. At Germantown an improvised cableway was set up, spanning the spillway gap, for the purpose of placing the concrete in the spillway bridge. At the other places it was placed by the same derrick that handled the material for the spillway weir. At Huffman and Taylorsville it was found advantageous to set up a tempor-

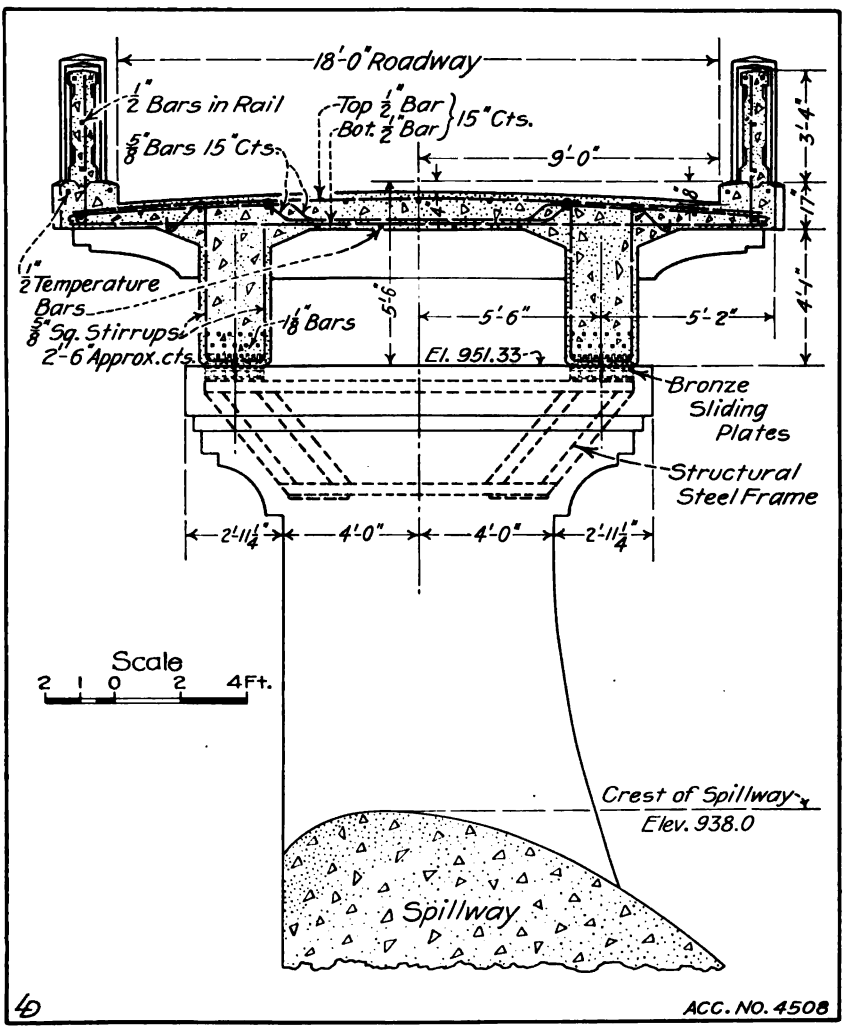


FIG 112—DETAILS OF CROSS SECTION OF LOCKINGTON SPILLWAY BRIDGE

Here the pier had to be considerably shorter than the width of the bridge, and the top of the pier was corbeled out as shown.

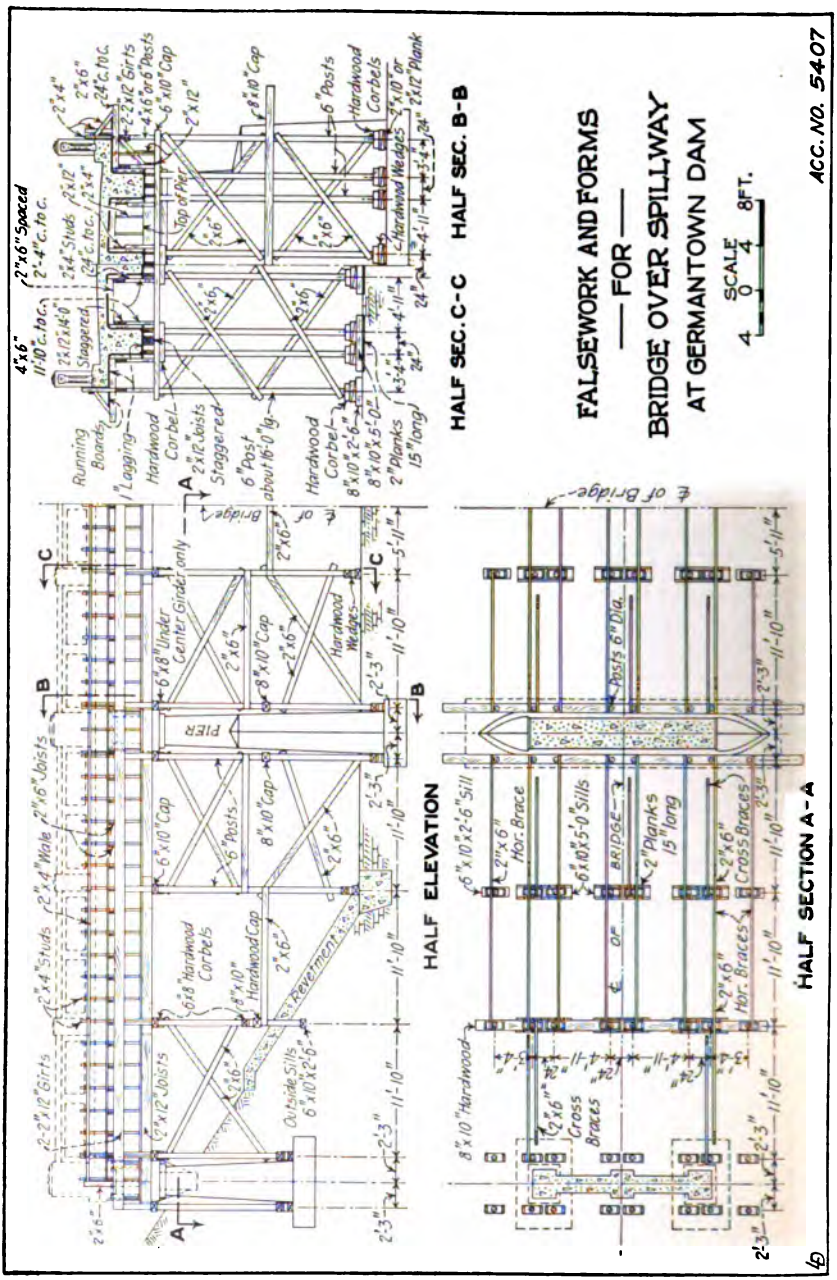


FIG. 113—DETAILS OF FALSE WORK AND FORMS FOR GERMANTOWN SPILLWAY BRIDGE

ary hopper at about the center of the span to be poured, which was kept filled with mixed concrete by the derrick; this was drawn out in small amounts and distributed by means of concrete buggies. Provision was made in each case for a wearing surface on the bridge floor, of rock asphalt or other bituminous material about one inch in thickness.

Naturally this concrete was expensive because of the relatively small quantities involved, the large amount of form work, and the quality of concrete required. Unit costs varied from $40 to $50 per cubic yard. Following are given the detailed costs of two of these jobs which are representative.

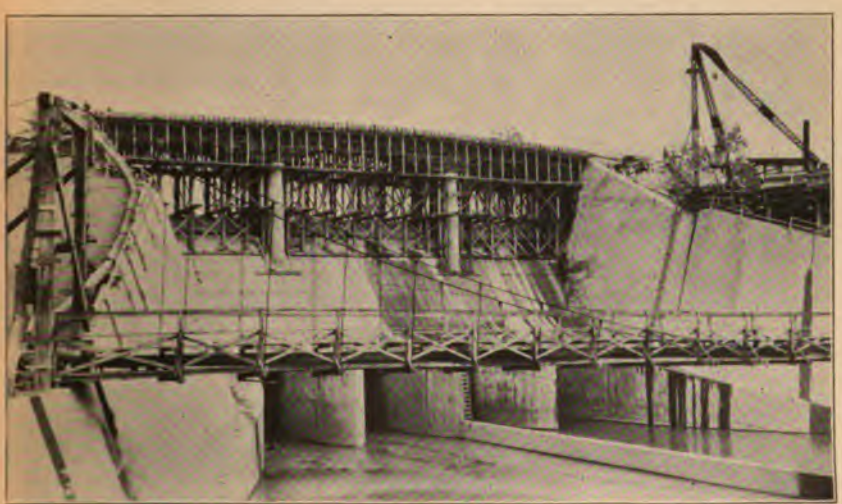


FIG. 114—VIEW OF FALSE WORK AND FORMS FOR TAYLORSVILLE SPILLWAY BRIDGE

Cost of Concrete in Highway Bridges at Germantown and Huffman Dams

Bridge at Germantown (453 cu. yds.)

		Cost Per Cu. Yd.
Superintendence and Overhead	$ 264.24	$ 0.59
Labor	11,947.33	26.38
Cement at $2.27 per barrel	1,697.39	3.74
Reinforcing Steel	1,432.96	3.16
Supplies	4,230.75	9.34
Plant Depreciation	117.14	.26
	$19,689.02	$43.47

Bridge at Huffman (403 cu. yds.)

		Cost Per Cu. Yd.
Superintendence and Overhead	$ 399.87	$ 0.99
Labor	10,513.66	26.09
Cement at $2.27 per barrel	1,657.10	4.11
Reinforcing Steel	1,128.58	2.80
Supplies	2,782.32	6.91
Plant Depreciation	399.84	.99
	$16,881.37	$41.89 per cu. yd.

The wage scale and the cost of materials for these jobs were about the same as those already given for the outlet and spillway structures at these dams. At the Germantown bridge the amount of cement per cubic yard of concrete was 1.7 barrels and at the Huffman bridge 1.8 barrels of cement per cubic yard were used.

FIG. 115—STEEL IN PLACE FOR FLOOR SYSTEM FOR ONE SECTION OF TAYLORSVILLE SPILLWAY BRIDGE

Black Street Bridge at Hamilton

Description: The old Black Street Bridge at Hamilton was destroyed during the 1913 flood. Subsequently a foot bridge was erected on the same site, but the City had not considered itself able to undertake the replacement of a highway bridge at this place until about the time that the work of the District

on the river channel was to be done. It was found that the interests of the District and the City would be served best by constructing a new bridge before the work of the District in this vicinity was undertaken. By such procedure the foot bridge could remain in service until the new bridge was opened to traffic, and the construction work on the bridge would not interfere with the other work of the District. Inasmuch as the District had the organization and equipment to do this work, an arrangement was made whereby the District would build the bridge and the cost would be assessed against the City.

FIG. 116—BLACK STREET BRIDGE AT HAMILTON

This is a seven-span reinforced concrete structure 708 feet long with a 28-ft. roadway and two 6-ft. sidewalks. It contains nearly 10,000 cu. yds. of concrete and 340 tons of reinforcing steel.

This is a seven-span reinforced concrete bridge 708 feet long with a 28-ft. roadway and two 6-ft. sidewalks, designed for a live load of 200 pounds per square foot plus one 20-ton truck and a 50-ton street car traveling in the same direction. It contains nearly 10,000 cubic yards of concrete, and 340 tons of reinforcing steel. (See Fig. 116.)

Construction Plant: .. In Chapter II the use of a dragline machine on the excavation for these bridge piers has already been described, also in the same chapter mention has been made of the pile driving for the foundation of these piers. For the placing of the concrete a Lidgerwood cableway was erected, spanning the river on the center line of the bridge. It was used extensively, not only for the placing of the concrete but also for handling and transporting form lumber, reinforcing steel, waterproofing materials, and all other such items as were

used in the construction of the bridge. For mixing the concrete a one-cubic yard Smith mixer was set up in the excavation for the east abutment directly under the line of the cableway. Details of any of this equipment may be had by reference to Chapter III. A batch hopper was installed directly above the mixer, and above that was a 33-cubic yard sand and gravel hopper. A 24-in gage track connected this sand and gravel hopper with the gravel storage pile, which had been placed on the river bank by a dragline machine which excavated for the piers. A small portable screening and washing plant was installed at this storage pile, and the prepared aggregate was hauled to the storage hopper over the mixer, in 1½-cubic yard cars hauled by a 3-ton gasoline locomotive. For the spandrel walls and railings, the concrete aggregates were purchased from a local gravel company which had recently started operations not far from the site. Close to the mixing plant a 3000-barrel cement shed was put up, and this was connected with the batch hopper by a plank runway. By extending the switch track belonging to one of the adjacent industries, it was possible to bring the cement up to this storage shed in carload lots .

Piers and Abutments: The pier footings are 62 feet in length, 20 to 24 feet wide, and vary in thickness from 6 feet at the sides to 9 feet in the center. The abutments are of the massive type commonly used for arch bridges. Before the cableway was erected, one of the abutments and a few of these footings were built by means of a small mixer set up on the bank of the excavation, the mixed concrete being handled in buckets by the dragline machine. As soon as the cableway was available, the use of the temporary mixing plant was discontinued. Each of the footings and abutments was built in three sections. A temporary hopper and chute was set up under the line of the cableway, for pouring the upstream and downstream sections. This hopper and chute were then picked up by the cableway and moved out of the way, and the middle section was poured direct. In the top of each footing were two rows of deep keyways and a large number of steel dowel bars to tie the shaft of the pier to the footing.

The pier shafts are 7 feet in thickness at the top and about 58 feet long. They are pointed, both upstream and downstream, and horizontal rustications were placed 3 feet apart on that part of the pier above low water. Forms were built in

place, consisting of 2-inch lagging against 2 x 12 studding spaced 24 inches apart. The forms were held together by ¾-in. rods spaced about 4 feet apart, both horizontally and vertically, which were protected by tin tubing and afterwards removed. These piers were poured in about 6 foot lifts, the hopper and chutes set on top of the forms under the line of the cableway being used for distribution of the material. All day's-work

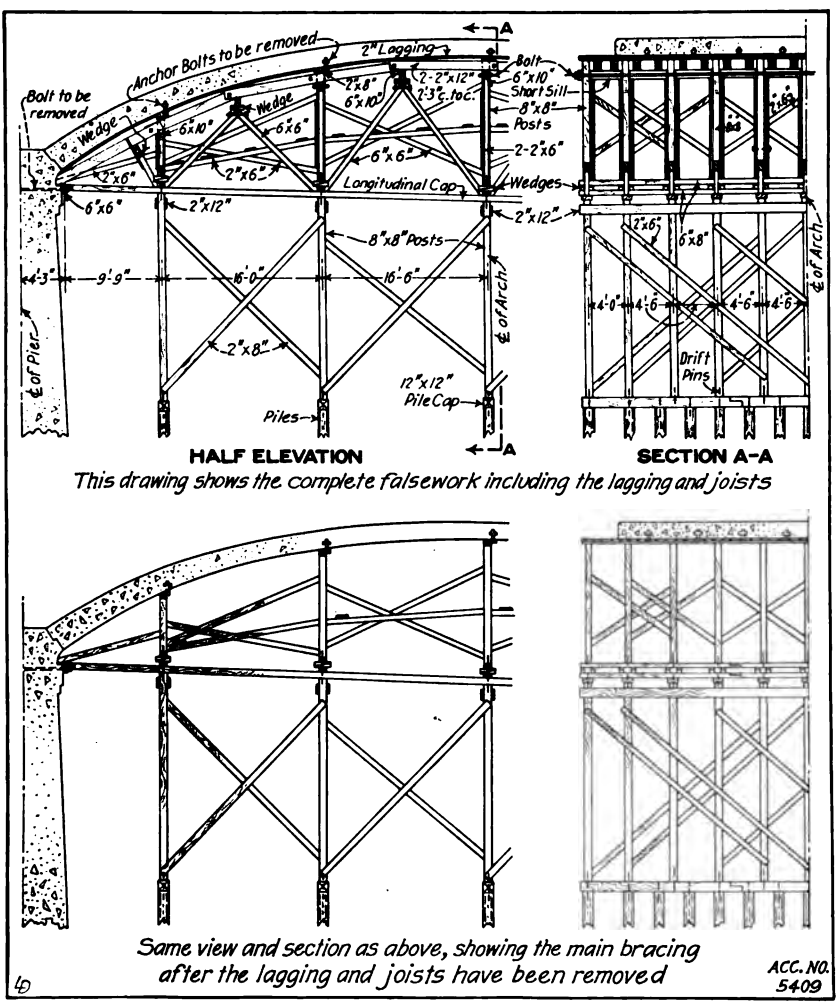


FIG. 117—DETAILS OF FALSE WORK FOR ARCHES OF BLACK STREET BRIDGE AT HAMILTON

The false work was so designed that most of it could be removed and still leave the posts in place to support the arch until the concrete was well hardened.

joints were made at rustications so as to render them practically invisible.

Several analyses of the bank run gravel at this place showed proportions of two parts gravel to one part sand with little variation. It was determined, therefore, to use the bank run material in the piers and abutments for this bridge. A mix of 5 sacks of cement per cubic yard of concrete was used except for the upstream nose of each pier, which was made of a richer mix having 6 sacks of cement per yard, because of the danger of wear from ice and drift. The concrete was mixed just wet enough to permit proper working in the forms; it was well tramped into place and carefully spaded.

FIG. 118—VIEW OF ONE SPAN OF COMPLETED FALSE WORK OF BLACK STREET BRIDGE IN PLACE

Arches: The false work for the arches was so designed that most of it could be removed and still leave the posts in place to support the arch until the concrete was well hardened. Fig. 117 shows these details. Fig. 118 is a picture of one span of the completed false work in place. Because the posts could be left in place to carry the load while the lagging, joists, and most of the bracing were removed, it was possible to use much of the lumber several times, and of all the lumber used on the entire work about half was salvaged in good condition, and either sold or used again on other work.

Each arch was poured in three longitudinal sections of equal width, each section being poured in one run. The middle section was poured first, keys being placed in both sides. As soon as the concrete had set sufficiently to permit removal of the

bulkheads, the outside sections were poured. To place the concrete, a hopper was set in the center of the arch, and chutes led to the ends of the section to be poured. Concrete was run alternately in each direction to balance the loading on the forms. It was found necessary to use top forms for a distance of 25 to 30 feet from the piers. These forms were built in sections about 6 feet long and placed one at a time as the concreting proceeded. Elevations were taken at frequent intervals while pouring, for the purpose of detecting any undue settlement that might occur. Settlement at the quarter points usually began

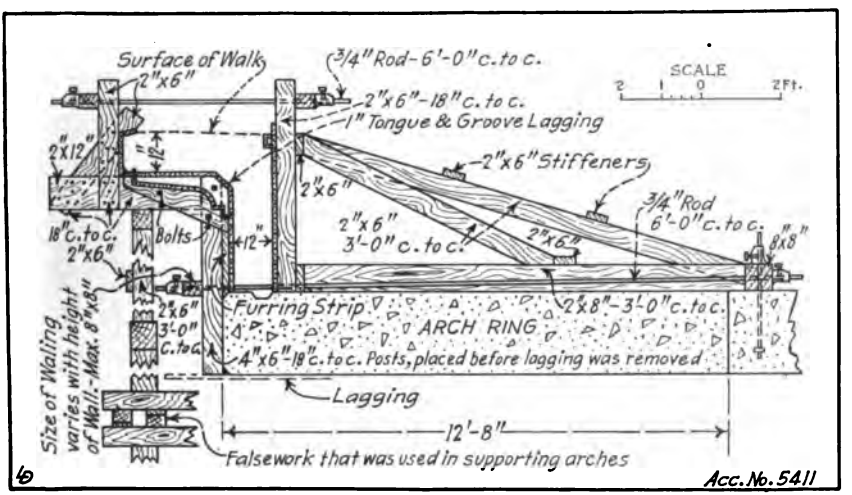


FIG. 119—FORMS AND SYSTEM OF BRACING FOR SPANDREL WALLS OF BLACK STREET BRIDGE

Outside forms were built first, most of the steel was placed next, and then the forms and bracing were completed.

when the concrete was within 5 to 10 feet from them, and in the center it did not occur until practically all of the concrete had been poured. The settlement was slight, in every case, and no more than had been provided for.

The mix used was approximately 1:2:4, using screened and washed sand and gravel, the largest of which would pass through a 3-inch circular opening. Steel stubs and keys were placed in the arch concrete to connect with the reinforcement for the spandrel walls and counterforts.

Spandrel Walls and Counterforts: The forms and system of bracing for spandrel walls are shown in Fig. 119. Outside forms

were built first, most of the steel was placed next, and then the forms and bracing were completed. Such procedure was necessary because with both forms in place the space between them was too narrow for a man to work in. Keys and steel bars were placed in the sidewalk to furnish suitable bonding with the railing. Each spandrel wall, from pier to pier, is divided into four sections, by expansion joints. These sections were poured alternately, two sections, one on each side of the arch, being poured from one setting of the hopper. Each section was run continuously from arch ring to top of sidewalk. Expansion joints were filled with a strip of asphalt and felt compound,

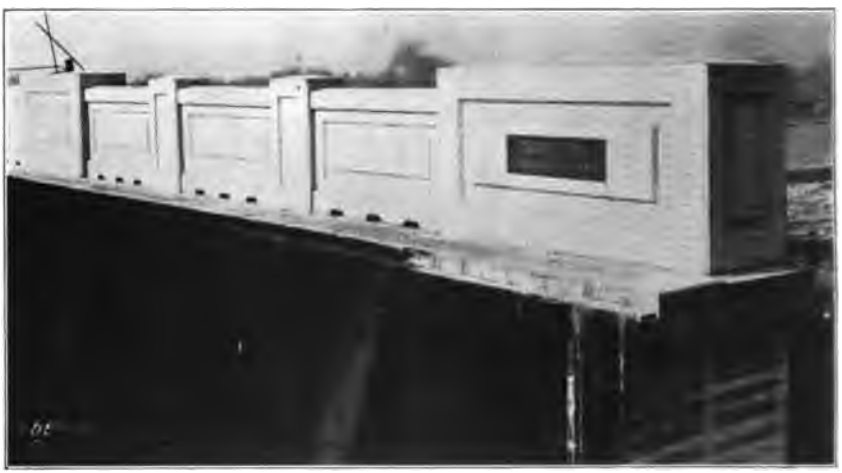


FIG. 120—RAILING ON BLACK STREET BRIDGE

Solid posts and panels were constructed, both reinforced with steel. They are strong enough to withstand the shock from a runaway automobile.

½-inch thick. Where railing posts were built over the expansion joints they were keyed on one side of the joint only, paper being placed under the other side to form a slip joint between the bottom of the post and the walk. Walks were finished with one inch of mortar colored with lamp black in the proportion of 2 pounds of lamp black to one barrel of cement.

The concrete for this part of the work was mixed about 1 : 1¾ : 3½ with the gravel limited in size to what would pass through a 1½-inch opening.

Pilasters: The pilasters over the piers were poured after the spandrel walls had been completed, and were separated from the latter by expansion joints. The form work and the methods of placing steel and concrete were similar to those used on the spandrel walls and require no special explanation. The mix

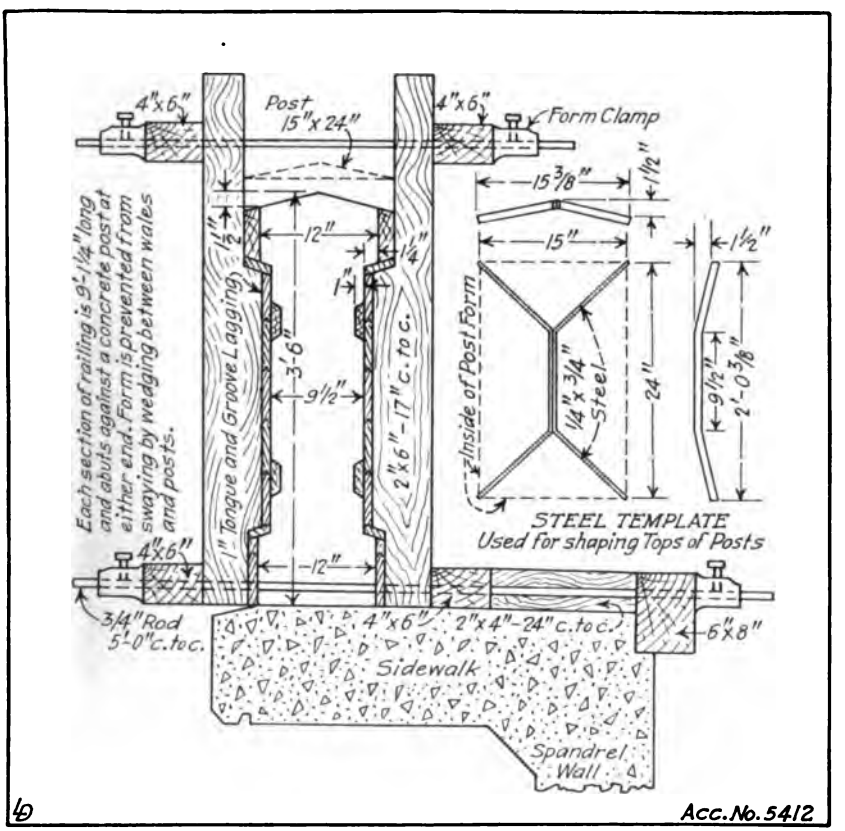


FIG. 121—DETAILS OF FORMS FOR PANELS AND POSTS OF RAILING ON BLACK STREET BRIDGE

The posts were concreted first, and after the forms had been removed the sections of railing were built between them. A steel template was used for shaping the tops of the posts and railings.

of concrete was the same. Fiber conduits were placed in the top of each pilaster to connect the lighting circuit with the lamp post to be placed over the pilaster.

Railings: The railing consisted of solid posts and panels as illustrated in Figs. 116 and 120. Both posts and panels were

reinforced with steel and are strong enough to withstand the shock from a runaway automobile, which is a requirement now recognized as necessary in bridge construction. The aim was to make a plain serviceable railing of good appearance, in keeping with the rest of the structure, without going to extremes in the way of decorative treatment. The posts were concreted first and after the forms had been removed the sections of railing were built between them. Fig 121 shows the construction of the forms. The mix used was 1:1½:3, sand and gravel being of the same quality as that used for the spandrel walls. The tops of posts and railings were finished with a steel trowel, no mortar being used. To obtain true lines for the ridges on posts and railings, frames were made of ¼-inch by ¾-inch iron, (shown in Fig. 121), and set in the top of each form as it was filled. The surface was troweled to this template which was then removed carefully and replaced with mortar.

After the forms were removed the railing was wetted and finished by rubbing with emery stones. When all irregularities had been smoothed off by the stones the entire railing was gone over with a damp brush. The openings shown at the bottom of the railing are for air circulation, to keep dust off the walks.

Waterproofing: Waterproofing was applied to the tops of all arches, tops of piers, to counterforts and spandrel walls below the top of the sidewalks. It consists of a bituminous oil primer coat and three coats of asphalt applied in alternation with two layers of cotton fabric. The primer was applied with brushes when the concrete was absolutely dry. It was found necessary to heat the primer and to thin it with gasoline except in very warm weather. Seven barrels, or 364 gallons of primer were used to cover the entire surface of 36,000 square feet. Asphalt was applied as soon as the primer had dried. It was melted in kettles such as are used in repairing streets, the kettles and aphalt being placed where wanted by means of the cableway. The temperature was maintained between 350 and 400 degrees Fahrenheit, usually at about 375 degrees. Sprinkling cans with the caps removed were used to distribute the asphalt. As it was poured on the surface it was spread with push brooms. Immediately following the first application, and while the asphalt was still hot, a strip of cotton fabric was applied, then another layer of asphalt and another strip of fabric, shingle fashion,

beginning at the pier and working up the arch. The fabric was lapped far enough so that all parts of the surface were covered with two layers of it. A final coat of asphalt was then applied. The same process was followed on the spandrel walls beginning at the bottom.

Costs: Costs of the various classes of concrete here described, are given in the following tabulation. During the period of this construction, labor rates varied from 35c to 46c per hour; carpenters were paid from 65c to 80c; carpenter helpers 40c to 52c; cableway runner 75c; with half time extra for more than 8 hours. Most of this work was carried on in 10-hour shifts which added 10% to the above base rates. The average cost of cement, delivered to the job, including freight and loss on sacks, was $2.57 per barrel. Reinforcing steel cost about $57.60 per ton, delivered. The price of lumber averaged about $35 to $40 per M. ft. B. M.

Cost of Concrete in Black Street Bridge

		Cost Per Cu. Yd.	Per Cent of Total
Piers and Abutments—6921 Cu. Yds.			
Superintendence and Overhead	$ 3,839.99	$ 0.55	3.2
Labor	59,239.62	8.55	49.8
Cement	20,380.10	2.94	17.1
Reinforcing Steel	2,250.20	0.32	1.9
Supplies	26,008.19	3.76	21.9
Plant Depreciation	7,251.20	1.04	6.1
	$118,969.30	$17.16	
Arches—1988 Cu. Yds.			
Superintendence and Overhead	$ 3,317.12	$ 1.67	3.9
Labor	39,864.82	20.01	47.0
Cement	7,758.83	3.90	9.1
Reinforcing Steel	13,717.60	6.90	16.1
Supplies	12,315.65	6.20	14.5
Plant Depreciation	7,967.92	4.01	9.4
	$ 84,941.94	$42.69	
Spandrel Walls—693 Cu. Yds.			
Superintendence and Overhead	$ 1,039.25	$ 1.50	2.7
Labor	24,995.79	36.14	64.5
Cement	3,423.24	4.95	8.8
Reinforcing Steel	3,177.10	4.48	8.0
Supplies	3,322.73	4.80	8.6
Plant Depreciation	2,892.54	4.17	7.4
	$38,850.65	$56.04	

Railing—191 Cu. Yds.

		Cost Per Cu. Yd.	Percent of Total
Superintendence and Overhead	$ 291.50	$ 1.53	2.5
Labor	9,115.29	47.73	76.6
Cement	1,012.58	5.32	8.5
Reinforcing Steel	483.36	2.53	4.0
Supplies	447.88	2.35	3.7
Plant Depreciation	558.77	2.92	4.7
	$11,909.38	$62.38	

(1415 lin. ft. @ $8.42 per lin. ft.)

Summary—

	Cu. Yds.	
Piers and Abutments	6,921	$118,969.30
Arches	1,988	84,941.94
Spandrel Walls	693	38,850.65
Railing	191	11,909.38
Totals and Average	9,793	$254,671.27

Average, $26.00 per cu. yd.

Waterproofing 36,086 sq. ft.—$4,545.11 = 12½c per sq. ft.

Adams Street Bridge at Troy

The unusual feature of constructing a new concrete bridge on top of a similar structure already in place, using the old bridge to support the forms for the new, was the most interesting thing about the reconstruction of the Adams Street Bridge at Troy. This work was an obligation of the County, but inasmuch as the District had the necessary organization and equipment, and was doing other work in the same locality, it was arranged for the District to do this work for the County, at an agreed price.

The old bridge was built in 1913, and consisted of four reinforced concrete arches. (See Fig. 122). The outward appearance of the structure was good, but the pier footings were inadequate and some of the concrete was in bad condition. The footings were so narrow that they could not take care of the unbalanced forces from the new arches with a sufficient margin for safety. They were also set practically on top of the river bed, and were placed on a foundation supposed to be composed of twelve-foot piling, but some of the piling proved to be only three feet long. The waterway under the bridge was wholly inadequate. The District calculated that it would be cheaper to reinforce the old piers and use them to carry the new bridge, to use the old arches to

FIG. 122-a—VIEW OF OLD ADAMS STREET BRIDGE AT TROY
This was a four-span reinforced concrete arch structure.

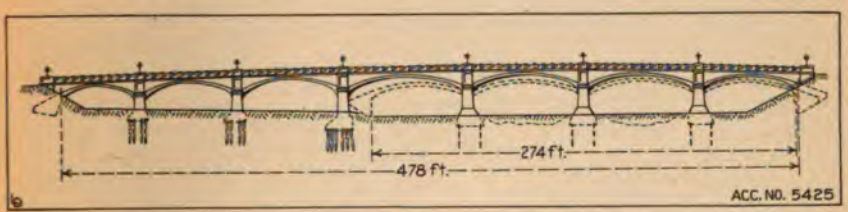


FIG. 122-b—ELEVATION SHOWING RELATION OF NEW ADAMS STREET BRIDGE TO OLD STRUCTURE
The old bridge had a total span of 274 feet, while the new one was built of seven spans, totaling 478 feet.

carry the falsework for the new, and to add the necessary extra arches, rather than to wreck the old bridge first, and build an entirely new one in its place.

Some very definite advantages were to be had by following this rather novel plan of utilizing the old bridge. The danger of floods washing out the timber falsework was done away with, as a clear waterway always remained under the bridge. Less timber was necessary in building the forms. Considerable saving in the volume of concrete was secured. The disadvantages were principally in the trouble of reinforcing and underpinning the old piers, and in the adventuring into the unknown which this plan necessitated. Without precedent to go by, the greatest care and thought was necessary to insure the success of each step in the work. See Fig. 123.

The deep water channel was under the old bridge, fore the building of the extension on the north end of the was carried on in the dry. Concrete sill were used to posts for the falsework. The superstructure and to

monly used for the piers, arches, and abutment of concrete bridges, were used for these three arches. No unexpected contingencies came up and no unusual construction methods were employed.

The spandrel walls, sidewalks, fill and paving were a considerable part of the dead load on the old bridge. They were removed before much work was done, in order that as little load as possible would be on the piers while they were being enlarged.

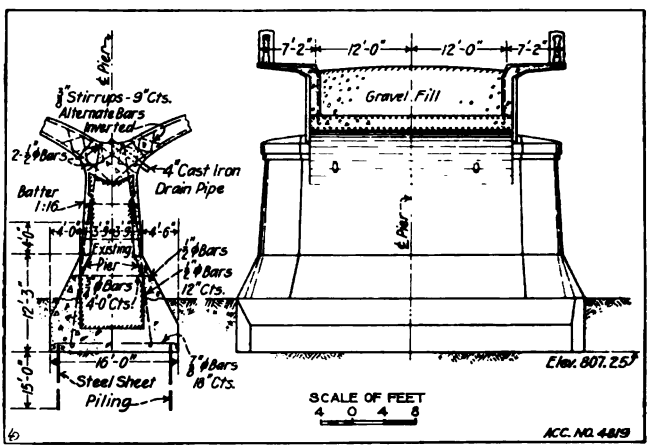


FIG. 123—REINFORCING OLD PIERS OF ADAMS STREET BRIDGE

The outline of the original pier is seen inside the final section. The footings were expanded to 16-ft. width at the bottom and steel sheet piling was driven to a depth of 15 feet below the bottom of the footings.

It was necessary to remove these carefully and to unload all arches simultaneously and uniformly, to avoid unbalanced forces on the piers. Most of the work was done by hand. Small shots of dynamite were used in breaking up the concrete wherever possible in order to lighten the hand work.

The first job in reconstructing the old bridge, was to enlarge and deepen the footings. The rebuilt footings are almost twice as wide as the old ones, and extend about three feet deeper into the river bed. A cofferdam of earth was thrown up around the pier, the water was pumped out, and the material around the footing excavated by hand. Taking one-third of the length of

the old footing at one time, the excavation was carried three feet below the bottom of the old footing, and then underneath. It was then that the condition of some of the old wooden piles was disclosed. Sheet steel piling fifteen feet long was driven one foot inside of the line of the new footing and projecting up one foot into the footing. Concrete was packed into the space underneath the old footing, and around the old piles out past the

FIG. 124—A PIER OF THE OLD ADAMS STREET BRIDGE READY FOR UNDERPINNING

One-third of the length of the old footing was handled at one time, the excavation was carried three feet below the bottom and then underneath. After the steel sheet piling was driven concrete was packed into the space underneath the old footing and around the old piles out past the line of the sheet piling.

line of the sheet piling. When the concrete was set, the next third was dug out, and the operation repeated. Figure 124 illustrates the procedure.

When the underpinning was all completed, holes were drilled through the old piers and the tie-rods placed through them. Then a jacket of reinforced concrete was placed entirely around

the old piers, the network of steel in the jacket being held in place by the tie-rods.

The reconstruction work on the piers and the south abutment did not turn out to be as difficult as was anticipated, although high water stopped the work a number of times. The difficulty of driving sheet piling in the restricted space under the bridge was overcome by the use of a small, compact steam hammer.

The north abutment had to be replaced by a new pier immediately north of it. As the excavation for the new pier removed

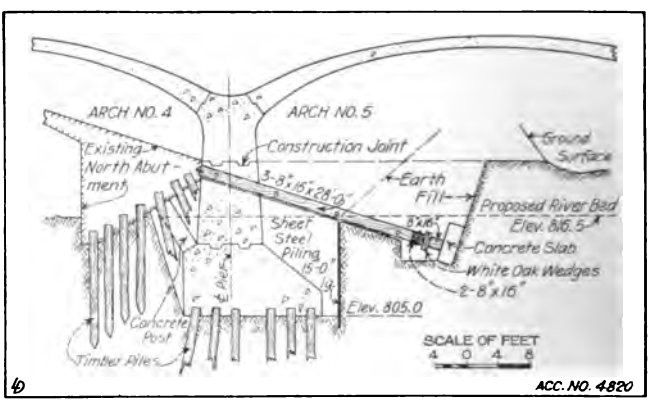


FIG. 125—METHOD OF BRACING OLD ABUTMENT OF ADAMS STREET BRIDGE

The north abutment had to be replaced by a new pier immediately north of it. In order to support the abutment until the bridge was taken down, heavy timbers, wedged against concrete footings, were placed in line with the direction of thrust and braced to prevent buckling. When the pier was poured the concrete was boxed around the timbers and the holes were filled in after the timbers were removed.

the bearing from behind the old abutment, some means of supplying a temporary support until the old bridge was taken down, was necessary. Figure 125 shows how this was accomplished. The heavy timbers, wedged against concrete footings, were placed in line with the direction of the thrust, and braced to prevent buckling. Then the excavating was done, and when the pier was poured, the concrete was boxed around the timbers. The holes so left were filled in after the completion of the bridge permitted the removal of the brace for the old abutment.

When all of the upper part of the old bridge except the arches was removed, and the piers had been reconstructed, the forms

for the new arches were built on top of the old bridge. The only difference from the forms ordinarily used on such work was in the posts. Instead of the long bents resting on piles or sills in the river bed, the posts were short, and rested on top of the old concrete arches, leaving an unobstructed waterway.

As the posts carrying the arch forms for the new spans on the north end rested on the ground, these arches could be poured in the usual way. But on the part of the new bridge that rested

FIG. 126—NEW AND OLD ARCH OF ADAMS STREET BRIDGE

After the new arch rings had set, preparations were made to remove the old ones just underneath. A trench was dug underneath the bridge deep enough to contain the old concrete. The old arches were then removed piecemeal, by use of small charges of dynamite and sledges and chisels. Special provision was made for balancing the thrust on the piers during removal of the arches.

on the old, a different method was necessary. To have poured each new arch complete would have caused an unbalanced loading on the piers that could have been dangerous. So each arch was poured in three longitudinal sections, one section being poured on all of the arches, and then another, and finally the third. By proceeding in this manner, the unbalanced thrust on the piers did not exceed a safe figure.

When the new arch rings had set, the most ticklish part of the job, the removal of the old arches, remained to be done. (See Fig. 126). Underneath the bridge a trench was dug, deep enough so that the old concrete falling into it would all be below the grade of the improved channel. Then one-inch holes were drilled through the old arches seven feet apart longitudinally and three feet apart laterally, and corresponding holes were cast into the new arches above. The holes were placed so that they

FIG. 127—LAST RIBS OF OLD ARCH DROPPING INTO THE RIVER AT ADAMS STREET BRIDGE

The network of reinforcing steel held the old concrete together and the steady nibbling reduced each arch to a narrow rib. These ribs were then cut and dropped into the river.

would be ready for a certain contingency. Due to the same difficulty about having too large an unbalanced thrust on the piers, the old arches had to either all come down at once, or else be removed uniformly. The practical difficulty of so managing affairs that all of the arches would let go and fall at the same moment was so great that this method was not seriously considered. So the piecemeal method was the one decided upon. The holes mentioned above were so arranged that bolts could

be run through them, taken up a bit, and the lower arches suspended from the upper ones, in case that during the operations hereafter described, the old arches showed any sign of failure. If this had happened, the suspended weight would have been very nearly equal on each side of each pier. The wrecking of the old arches was much easier than had been expected. Scaffolding was suspended from the upper arches by means of rods run through the holes mentioned above, and workmen using sledges, small charges of dynamite, and chisels, began taking out little bites from all of the arches, working at about the same rate on all of them.

The net-work of reinforcing steel held the old concrete together very well, and the steady nibbling reduced each arch to

FIG. 128—COMPLETED ADAMS STREET BRIDGE AT TROY

a very narrow rib, without any necessity arising of suspending any of the old structure from the new. These last narrow ribs were cut and dropped without incident. (See Fig. 127) .

The construction plant was not elaborate. A derrick dug the gravel needed in addition to that already piled up on the site by the dragline. A portable gravel-washing and screening plant prepared the material for use. The aggregates were loaded into the Smith one-yard mixer by gravity, and the concrete was transported to the work in side-dump concrete cars that ran over a narrow-gage track, up over the levee, and across the river on top of the forms. It was not necessary to hoist any of the concrete into position. A gasoline locomotive did the hauling.

Fig. 128 shows the completed bridge. The type of railing is similar to that used on the Black Street Bridge at Hamilton and illustrated in Fig. 120, namely, a substantial solid panel and post design with the minimum amount of ornamentation.

Concrete Tablets at the Dams

One of the most unique and satisfactory concrete jobs on the whole District work was the construction of a large tablet at each of the dams, giving brief information regarding the structure and its purpose, for the benefit of visitors and passing motorists. The details of these tablets are shown in Fig. 129. The wording of each of the five was the same, except as to the data regarding the structure itself, i. e., the whole of the first line, and the figures in the last three.

These dams during their construction and after completion were objects of interest not only to engineers and construction men, but to the public at large. Thousands of tourists visited them each season, and because the National Highway, one of the main east and west thoroughfares across this part of the country, passes over the tops of two of the dams, many tourists were brought to them undesignedly, and immediately became curious as to their purpose. For this reason it was determined to erect a suitable sign at each dam, which would give very briefly the necessary information. Because of congestion of traffic on Sundays, holidays, and other certain times, it was decided that the inscriptions on these signs should be large enough so that they could be read by passing motorists without stopping. This required a large sign even to contain a very brief description. Some kind of permanent construction was desired. Execution in bronze or granite was ruled out because of excessive cost. Cast stone would not do because of the excessive cost of handling, hauling, and breakage that would be involved in this case. It was finally determined to attempt the construction in concrete, with the assistance and advice of a competent architectural designer. Fig. 129 shows the essential details of design.

The minimum size letter, such as appears on the bottom line of the inscription, was fixed at 3½ inches high, and the maximum height for the top line was fixed at 8½ inches, so that it could be read at considerable distance. All letters were incised as shown, so as to cause sharp shadows. No reliable data being available as to how best to secure a strong, dense, stainless face, a series of experiments were conducted with various materials and various combinations.

Sample casts were first made using White Atlas Stainless Cement with proportions of white lake sand varying from 1:1 to 1:3. Each such mix was made of three consistencies—very

plastic, stiff plastic and semi-dry (crumbly). Similar test blocks were made of White Atlas and brown river sand. Other test blocks were made with marble dust in place of sand. Other test blocks were made of white sand and common Portland Cement. Others were made with part White Atlas and part common Portland cements and white sand, the proportions of White and Portland cements being varied in different test blocks.

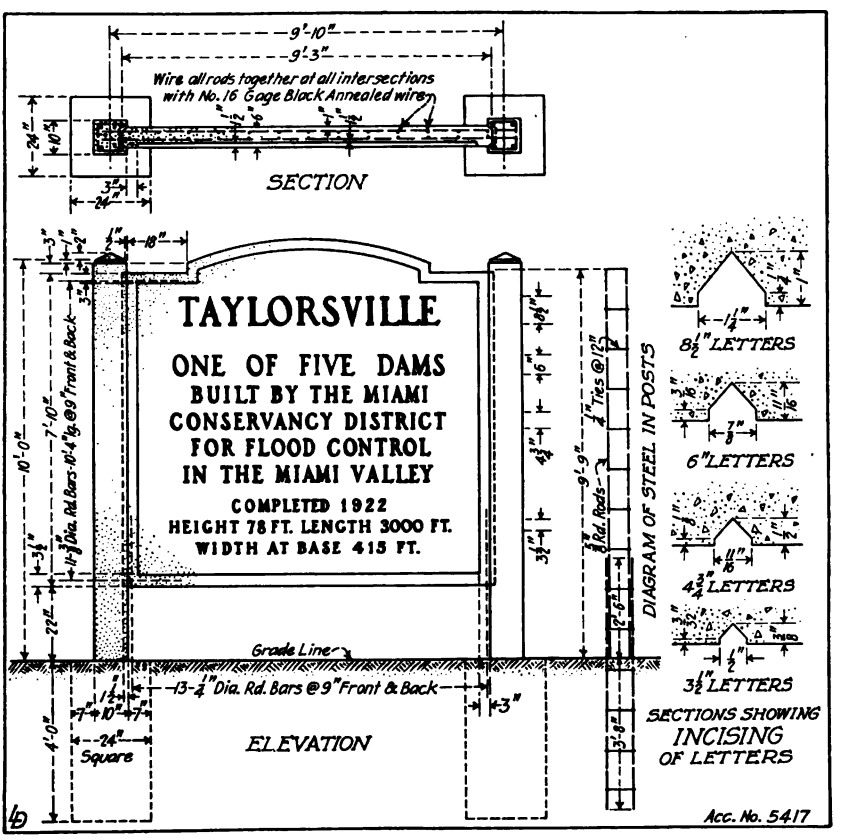


FIG. 129—DETAILS OF CONCRETE TABLET ERECTED AT EACH DAM

Considerable experimenting was necessary before a satisfactory method of construction and type of mix was evolved for these tablets. Aluminum patterns were used for the letters. The tablets were cast face down, with a mortar for the face mix about 2 inches thick made up of 2 parts white Atlas cement, 1 part Portland cement and 6 parts white lake sand. The backing was of 1:1½:3 mix Portland cement concrete reinforced with ⅜-in. rods. Decorators' muslin was placed on the form, muslin face to the concrete.

These samples were all made according to laboratory practice, of measured parts, kneaded to uniform consistency and kept out of doors under moist cloths for two weeks before removing form, and were seasoned for one week thereafter. They were then inspected for general appearance, color, density of face and body, and texture. Then they were sounded by suspending and ringing with an iron bar. A series were rung in overlapping succession and the best selections laid aside. Samples were then tested for impact, scratching, tensile strength, and fractures, and inspected for density and general appearance. It was found that in every case the mixes of medium (stiff) plastic consistency were densest and strongest. The mix of one part common Portland and two parts White Atlas to six parts white sand were as strong as any other mix and gave the most uniform color and texture. The richer mixes showed crazing, the leaner mixes lacked density, and the greater parts of common Portland gave irregular color effects.

At this point the selection of the mix became limited by another important factor, the removal of the patterns forming the incised lettering. Because of the large total number of letters required for five tablets the comparative cost was considered first. It was assumed that if wood or composition letters were used, they would either become distorted by moisture or pressure in the casting process or, if good casts were had, they would probably become damaged through being obliged to remain in and under the cast during the seasoning period, and therefore would be unfit for further use. Comparative estimates were had for the making of five full sets of wood letters and for the furnishing of one full set of brass or aluminum letters. The cost being approximately the same, aluminum was selected, and thus the construction program was limited to one tablet at a time, at intervals of about two weeks, to permit the removal and re-use of the letter forms.

These aluminum letter forms were used in all test blocks. The first were uncoated and could not be removed, after two weeks, without damaging the edge of the incision. This was due to adhesion between the cement and the metal. Next they were dipped quickly in hot paraffine, resulting in a coating 1/16-in. thick. This was found to be too thick a coat, resulting in distorted cross-section. This coating also peeled off readily in applying the letters to the form. It was found that holding the

metal in the hot paraffine for a brief time resulted in heating the body of the metal so that a thinner coat adhered. This too was slightly fragile at sharp points on the letters, and in addition the cement broke through the protective coating and formed adhesions. On stripping, the incised surfaces were found to be irregular and the letter outlines distorted. Shellac was next tried but was found to encourage adhesion. At this point it was proposed to increase the draught of the sides of the letters for a distance of 3/16 in. from the face. This discussion developed the fact that these vertical faces were essential to produce a well defined shadow line, and that a slight increase in the draught of these sides reduced the legibility of the letters to a marked degree.

Progressing from the experiments with paraffine, the letters were next dipped in hot beeswax and it was found that when left submerged for a brief period (10 to 20 seconds) they retained a thin coat which resisted the placing and setting of the mortar, and permitted the free removal of the patterns without damage to either tablet or pattern. Further casts made over these coated patterns showed that the medium plastic consistency, which gave the best structural results, did not give satisfactory finished surfaces for the incisions. The beeswax presented an impervious surface to which water clung without absorbing, and when stripped the surfaces were pitted and porous. Moisture contact was reduced until a crumbling consistency was obtained, which, when sufficiently tamped, gave a highly satisfactory surface within the incision.

Another item for consideration was the surface of the form proper. The first experimental casts were made on a form of jointed white pine, glued and battened and coated with paraffine oil. The finished cast reproduced minutely the grain of the wood, and the glued joints were so distorted by the presence of moisture as to mark the surface. All methods which were successful in removing these marks involved partial destruction of the dense face coating. Also, the paraffine oil reduced the absorption of the surface so that small globules of water were left to evaporate, which resulted in pitting. This occurred even with the crumbly mixture and, since the flat surface lacked the drainage which the raised letter patterns had, it appeared that the form proper had to have some absorbent quality.

A heavy quality of shrunken muslin was stretched over the wood form. This had the proper absorbent quality and added to the uniformity of the finished surface, but in large sheets was found, when wetted, to stretch and buckle between the letters which were nailed through it, leaving wrinkles impressed in the face. It was then found that a form of filled and prepared decorators muslin, similar to Sanitas, when applied with the muslin face up, (to the concrete), gave the desired surface and absorption, and possessed sufficient body to prevent distortion.

These several factors being determined, the execution was fairly simple. The muslin was applied to the form and the surface penciled in modules for placing the letters. The dipped letters were then applied, using 3d common wire nails to secure them to the forms. The form was placed solidly and securely and level at its approximate location. The mortar (crumbling plastic) was applied 2-in. deep by hand, placed around the letters with the fingers, and tamped thoroughly with a wood mallet or gavel. This process was done as rapidly as possible, after which was added four inches of concrete, 1:1½:3 mix, reinforced with ⅜-in. rods 6-in. apart each way. The whole was tamped thoroughly and covered with four layers of wet burlap, which was kept wet continuously for two weeks. At the end of this time a gin pole and chain block were set up, heavy rope slings were placed around the slab, and it was raised to a vertical position and braced. The forms were then removed and moved on to the next site. Supporting columns extending well into the ground were formed and poured around the edges of the tablet, forming a permanent and monolithic whole. After the surfaces were thoroughly air seasoned, they were brushed with a stiff bristle brush, thoroughly saturated, and a thin grout of White Atlas cement brushed lightly on. After about fifteen minutes, when this began to turn light, indicating partial evaporation, but before initial set began, the surface was rubbed briskly with excelsior to remove all cement not absorbed into the pores of the face. This method was intended to form a natural integral filler for the pores which would reduce absorption and consequent frost action, and to avoid crazing.

Fig. 130 is a view of one of the completed tablets. The results obtained appear to be wholly successful. The tablets were completed in November, 1923, and have been subjected to two winters and one summer season. No settlements or

fractures due to external stresses are apparent. No shrinkage cracks or crazes are observed, and the uniform resistance to temperature variations indicates comparative freedom from internal stresses. Minute crazing is observed in some of the supporting columns, probably due to the fact that the architectural design limited these to 10-in. square. Since they were to be supporting shafts rising 12 feet, they contained four ½-in. rods laced, and since they had to be cast monolithically, the lack

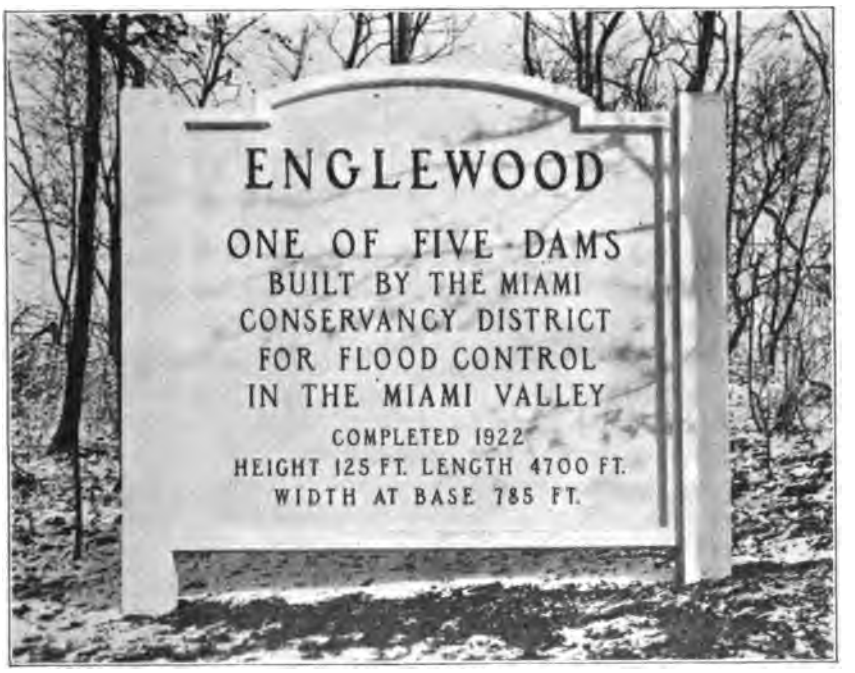


FIG. 130—CONCRETE TABLET AT ENGLEWOOD DAM

These tablets have been subjected to two winters and one summer season and no settlements or fractures due to external stresses are apparent; nor are there any shrinkage crazes or cracks or traces of erosion.

of space in the form required a mortar free from coarse aggregate and a wet plastic mix which would fill the form free from voids. There is no doubt that a leaner, coarser and drier mix would have given better structural qualities had the pouring space permitted. Particular note has been taken as to erosion from wind or frost, but no trace of such has yet been observed.

The surfaces show no signs of weather stain and appear to be permanently and uniformly grey white. The incised lettering, with vertical draught part way back from the face, gave fairly satisfactory results, but legibility has been greatly improved by the simple expedient of touching them up with a thin, black, waterproof paint.

All of the tablets have been subjected to minor vandalism, principally the impact of stones thrown by small boys. In a few instances small caliber rifles have been used. Neither has had any damaging effect. In one instance a shotgun was discharged at close range and a slight abrasion resulted, but the uniform density of the mortar was such that the abrasion is not apparent except at close range, and is not disfiguring.

The work appears to justify the hope of its designers that concrete can be successfully and economically employed as a substantial, permanent and pleasing architectural medium, but the results here obtained are believed to depend not only on the thorough and careful consideration given to the selection of materials and methods, but in larger part to the extraordinary patience and accuracy of the workmen in executing the work.

The organization for the construction of this work was interesting, because of its unusual nature. In the first place while there were five similar casts to be made, only one set of metal letter forms was available, and casts could only be made successively. Moreover, the five sites were several miles from Dayton and distant from each other from six to fifty miles. The secret of keeping the costs within reasonable limits, was to organize a small gang of highly specialized men who could move around from one job to another, and handle each so as to minimize waste of time, and still give the concrete plenty of time to season after each step in construction.

The superintendent chosen was, in fact, a contractor of high intelligence, who was genuinely interested in the technical results and in the costs. He devoted his entire time to the work and became also the chief mechanic, personally performing the delicate parts of the work, including dipping the patterns, placing the muslin face on the form, placing the metal patterns, placing the mortar with his hands, and wetting the casts daily. His only regular assistant was a skilled carpenter, and he employed two laborers for brief intervals to mix the coarse aggregate and to assist in hoisting and placing the casts.

The form was set up at the first site and the cast made. The form was then engaged for two weeks before the next cast could be made. In that time the materials were assembled at the second site; the supporting frame and form was built ready to set in the form panel carrying the letters; the column forms and laced reinforcing were prepared for the first site and the column footings excavated and cast. In addition, the first cast was visited and wetted twice daily, and the location was selected for the third site. This consumed the two weeks for the two men engaged. When the lettered form was removed from the first cast it was hoisted in slings, and left until the second cast was made. Two days were required to remove the form, transport it to the second site, rearrange the wording, recoat the letters, and make the cast. The men then placed the first cast in position, placed the column forms and reinforcement, and cast the columns. They then prepared the column forms, reinforcements, and footings for the second cast, moved the bracing and form supports from the first site to the third, and prepared there for the reception of the lettered form, accumulated material at the fourth and fifth sites, wetted the second cast twice daily.

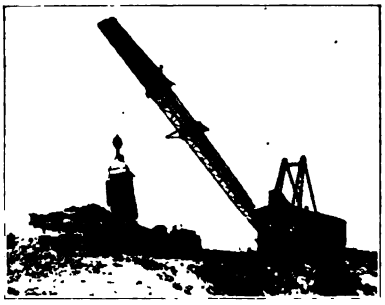
It was intended that during these intervals the finishing of the surfaces would be carried on, but with each additional cast the average daily transportation cut down the actual working hours so that only a small portion of finishing was done until the last cast was made.

The following figures tell the story of the costs. Labor, of course, was a high percentage, and materials a very low percentage, of the total. Transportation was a big item, and so was the cost of the letter forms. Considering the results, and the unusual nature of the work, the job is rated as wholly satisfactory.

Cost of Concrete Tablets at Dams

	Five Tablets	Each	Per Cent of Total
Supervision and Overhead	$ 476.86	$ 95.37	9.1
Labor	2,530.67	506.13	48.3
Transportation (men and materials)	676.00	135.20	12.9
Materials (cement aggregates, steel)	363.47	72.69	6.9
Aluminum forms for letters	825.00	165.00	15.7
Other supplies (lumber, bolts, etc.)	373.47	74.70	7.1
	$5,245.47	$1,049.09	100.0

These costs include all experimental work, which amounted to approximately $350.00.



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